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Making Camp Life Agreeable Progressive contractors and construction engineers are no longer satisfied with a camp that is merely sanitary and well maintained, though in smaller organizations even these first requisites for tolerable life in the field are commonly neglected While special attention to camp planning will be well repaid in more efficient service in every part of the country, it is specially needed in isolated districts where the men are cut off from all recreation and amusements, unless these are provided in camp. An example of what can be done to make camp life agreeable is described in the article on page 116. Arrowrock is 20 miles from any sizable town, from which the conclusion might naturally be drawn that its life would be dull and monotonous. And while it may lack the gayety of large cities, a baseball diamond, a tennis court, pool tables, moving-picture shows, a library and lectures offer a variety sufficient to satisfy almost any one. As is to be expected where so much consideration has been paid to the mental and social sides, the provisions for the physical well-being of the camp are of the highest order. The story is well worth reading in detail even though one is not charged with the laying out and maintenance of a construction camp.

Ultra-Violet Ray Sterilization From the first announcement that water could be effectively sterilized by the use of ultra-violet rays the process has received much attention, for there is a fascination about the method that places it in a class by itself. Satisfactory results have been obtained on a small scale for some time, but the improvements described by Dr. von Recklinghausen on page 121 indicate that the process may now be extended to much larger plants. The chief improvement is in the lamp itself, so that with a given amount of energy the new lamp is about five times as effective as the old. The new lamp operates on a 500-volt circuit and consumes 3 amperes. Practically all the light enters the water, while formerly only about 60 per cent was used. One of the important points made by Dr. von Recklinghausen is that rapid filtration before exposure to the light is necessary in order that there may be efficient sterilization. Suspended matter interferes with the rays and prevents effective exposure of all particles of the water to the sterilizing action. As was to be expected, many difficulties have been encountered but apparently have been satisfactorily solved—so far at least as bacterial efficiency is concerned. Data as to costs, in comparison with other processes, are now needed to indicate the applicability of the process to other installations.

Practical Work for Instructors The employment of twenty-eight members of the instructional staffs of various engineering colleges in the shops of one of the large electrical companies, as recently recounted in a news item from Pittsburgh, serves to emphasize, what is already well known, that there is each year closer contact in engineering matters between the colleges and the school of experience. In almost every line of engineering, when work is plentiful, large numbers of instructors and professors avail themselves of the summer vacation to learn at first hand how theory and laboratory results stand up under the pressure of execution—when decisions mean not scholastic standing but dollars and cents. While much credit is due to the instructors for their attitude in engaging in such vacation work, tribute must also be paid to the manufacturers, railroads and engineers who are sufficiently appreciative of the favorable influence of these men on their work and organizations to suffer some inconvenience in making temporary places for them. Undoubtedly at times these men come along when additional help is needed, but often the places must be created or special work taken up to accommodate them. It would be idle to discuss here the benefits to the teacher's future work of applying his theory and his knowledge acquired under laboratory conditions to practical problems. The result is a higher appreciation by the instructor of the difficulties confronting the man in the field and a more tolerant attitude toward the short cuts and often crude methods employed by the practitioner. Moreover, where his experience has demonstrated that the application of more highly theoretical methods would result in undoubted advantages and economy these methods will be thrown into higher relief and his confidence in them strengthened.

Profit in Garbage Reduction New York City recently received bids for the collection and disposal of its garbage which showed such a great discrepancy that to the popular mind only one course was open —to award the contract to the bidder who offered to pay for the privilege of removing the garbage rather than to the company which demanded that it be paid for the work. The former offered $87,500 per year for the first three years for the privilege, with an increase to $112,500 per year for each of the two succeeding years. The second bidder (the present contractor) demanded the payment of $130,000 per year for serving the city. The difference to the city would, in five years, amount to $1,137,500. In explaining that there may well be hesitancy in awarding the contract to the lower bidder the commissioner of street cleaning pointed out that an investment of about $1,000,000 in plant was required, in addition to the furnishing of a $100,000 bond and a $40,000 cash deposit. In the face of these heavy financial requirements the city did well, as the commissioner pointed out, to ponder the effect of inability to handle the work successfully—for that would probably mean the accumulation of garbage in the streets, resulting either in extraordinary emergency measures or in menace to the public health. But more interesting than these main considerations is the statement of the commissioner that the profits in garbage collection and disposal are very largely dependent upon the disposition of householders and hotel proprietors. The profits are now largely cut down because the valuable parts of the garbage—bones and fat—are sold to soap makers, leaving only the unprofitable part for the city to remove. Even though the returns in individual cases from the sale of these materials are small, it is unlikely that the commissioner's appeal for a public-spirited attitude in foregoing these returns will have any effect. It is probable, too, though the commissioner does not raise the point, that the city has no way of stopping the present practice. The matter is well worth while for other cities to bear in mind in taking up the matter of garbage reduction, for evidently statistics which are silent on this point are likely to be misleading.

The Binghamton Disaster Progress toward adequate provision against loss of life by fire seems to be slow and what advances are made are largely the aftermath of horrifying disasters. One would expect that the Asch Building fire in New York City, in which 147 persons perished, would have resulted in legislation covering all classes of structures. It did, in fact, result in decided progress in safety legislation in New York State, but the argument accepted by the framers of the law was faulty. The greatest danger, it was apparent, was in tall buildings, and so far as they are concerned regulations that are quite satisfactory have been put into effect. The loss of twenty-five lives and the injury of forty employees in a four-story factory-building fire in Newark seemingly did not influence the legislators, for inclosed fireproof stairways or adequate steel stair-towers protected from possible flames from windows were not required for small buildings. It is reported that this provision was defeated by the manufacturers up state, who claimed that the cost would be prohibitive. But whatever the influence or the reason, a demonstration of their need was attended by the loss of forty lives in the fire in the Binghamton Clothing Company's building at Binghamton, N. Y., on July 22. The stairway was not fireproof, the flames swept across it, egress was cut off and the outside fire escapes proved inadequate. Half-way safety measures, confined largely to fire drills, are criminal in the light of the death toll in recent factory fires. The pocketbook and influence are allowed to stand in the way of adequate measures. Inclosed fireproof stair-towers and steel towers which cannot be reached by flames from lower stories have the practically universal approval of those who have studied the problem. The work for the legislators is clearly pointed out.

Drivel about Hetch Hetchy Scare headlines have been the rule during the past week in Eastern newspapers regarding the utilization of the Hetch Hetchy Valley for the supply of water to San Francisco. "A National Park Threatened" and "The Thirst for Hetch Hetchy" are samples of the heads used. Most of the items are based on the appeal of Mr. John Muir, which when carefully read will in itself convince most men that the "nature lovers" are carrying their objections to extremes. A perusal of the first part of Mr. Muir's argument brings up visions of the invasion of a mighty domain and the destruction of a beauty spot unique and unparalleled. It is disappointing, therefore, to learn toward the end of the appeal that the scenery that has stirred up his fervor is comprised in a valley "about 3½ miles long and of a width varying from one-quarter to three-quarters of a mile." It is, in fact, but a small part of the Yosemite National Park, and not by any means its chief scenic feature. Moreover, there are many who hold that the formation of a lake in the valley, far from destroying its beauty, will actually enhance it. Coming down to facts, however, the situation is so simple that only one conclusion is warranted—that Congress should pass the legislation enabling San Francisco to proceed with the work. The entire question of the water supply of San Francisco was investigated thoroughly by a competent engineering organization, which reported in favor of Hetch Hetchy. Leaving aside the possibility of going to other sources, whether more costly or not, it is evidently absurd to employ competent experts and then to ignore their conclusions. Moreover, the Hetch Hetchy plan has the approval of the Secretaries of Agriculture and the Interior, the Army Board of Advisory Engineers, the Directors of the Reclamation Service and the Geological Survey and the Chief Forester. At the outset strong opposition came from two quarters—the nature lovers and the Modesto-Turlock irrigation interests. The latter have been satisfied in the Raker bill, authorizing San Francisco to use Hetch Hetchy, leaving only the "scenery people" as opponents of the measure. They will not cease their opposition under any circumstances, and it is hoped, therefore, that the members of Congress will ignore their appeals. Against their flimsy objections are thorough engineering reports and the endorsement of high government officials.

Engineering Standards and Architectural Requirements

The development of the steel-cage building, with its long spans, heavy load and complicated mechanical problems, has made the association with architects of structural, mechanical and electrical engineers the prevailing practice on large buildings. Such an arrangement is, in fact, absolutely necessary for economical design, for with present requirements in large hotel, office, theater and loft buildings the architect cannot expect to be a master of all of the specialties entering into the completed structure. Co-operation of this sort is generally secured by associating independent consulting engineers with the architect. For this service the owner either pays the architect a lump sum sufficiently large to retain specialists for the different classes of work or the architect is engaged solely for the strictly architectural features and supervision of the work, and separate retainers are given to the specialists. When an architect's practice is so extensive that he is enabled to command the continual services of thoroughly trained specialists, then they may well belong to a common organization.

But while such association of engineers with architects is the rule on large projects, it is not as fully appreciated that association of this character, in order to result in effective co-operation, often requires the subordination of the strictly engineering ideals of economy and efficiency to the artistic or architectural standards in the mind of the master craftsman—in this case the architect. Even the most successful structural engineers have learned that many purely theoretical requirements and advantages of their designs may safely and even profitably be sacrificed to architectural considerations.

For example, the cost of the structural steel in a tall office building may equal 10 or 12 per cent of the total cost of the building and its equipment. The imposition of very difficult restrictions or uneconomical requirements will hardly increase this cost more than 25 per cent above the minimum for which the structural steel can be designed on the basis of maximum efficiency and economy. This will amount to only 2 or 3 per cent of the total final cost of the building, and may safely be negligible if attractiveness, utility or convenience demands it.

The provision of a 40-ft. clear width in the ballroom or dining-room of a large hotel may involve the use of 100 tons more of structural steel, and thus cost perhaps $6,000 more than the use of short spans over the same space. This sum, however, when equated with the capitalized rental from the enlarged space or with its greater attractiveness may prove to be an advantageous expenditure. In the same way the use of heavy shallow and uneconomical girders to secure headroom or provide clearance may be justified by the benefits derived therefrom. In other cases, as where a column spacing of, say, 12 ft. is considered to give too shallow beams and girders to provide the best results in wind bracing, it might be very unwise to insist on increasing the span if that is predetermined by important building or operating considerations.

In other words, the lesser evil or the greater good should be the prevailing motive for the decision. A successful and experienced structural engineer summarizes the condition by saying that "the greatest advantage to the building and its functions as a whole should be considered." This attitude has, moreover, another very decided advantage, for when constructions that are really dangerous or very objectionable are called for the architect is the more willing to respect the engineer's objections. A spirit of compromise, which nevertheless does not waive fundamentals, is a wonderful inspirer of confidence.

Educational Work in Highway Engineering

Much publicity has recently been given to the proposal by former Senator Jonathan Bourne, Jr., for the establishment of a national academy for the training of highway engineers. While the proposal was made in conjunction with a special plan of joint highway financing by the state and the nation, which has little likelihood of being accepted, it nevertheless offers food for thought to those sincerely interested in raising the standards of highway construction. As is generally known, several of the universities have taken a special interest in highway training during the past year, while the Office of Public Roads has had a course of instruction since 1905.

Civil engineering students are appointed to the course in the Office of Public Roads through civil-service examination, and during the first year are given a thorough training in all branches of highway work, both in the field and in the laboratory. At the end of the first year, if the students prove worthy and the needs of the service justify it, they are promoted to the position of junior highway engineer. At the close of the second year they are eligible for further promotion to the grade of highway engineer and ultimately to the position of senior highway engineer. Candidates, however, are not eligible for examination unless they have been graduated in civil engineering from one of the leading engineering colleges.

From 1905 to date fifty appointments have been made under this plan, and of the appointees thirty-three are still engaged in highway work, twenty-one of them in the Office of Public Roads. Twelve of those who have resigned accepted positions with county and state highway organizations or are in some form of highway work. Seventeen of those who resigned did not take up highway work, but entered other engineering departments of the government or engaged in engineering with private organizations.

While the plan has given excellent results, the practice of permitting engineers to resign is detrimental to the office itself, for those who leave are frequently among the best men in the service. Nevertheless, the benefits accruing to the states and counties through the distribution of trained men to all sections of the country are considered sufficient to vindicate the wisdom of the policy.

While good work is thus being done by the Federal government through this course, the number of trained civil engineers needed in highway work while large must always be small in proportion to the number of superintendents and minor officials required for county and state highway work. It is manifest, therefore, that there is a large field for instructional work along these lines, and engineering colleges in every state should be prepared to meet the demand. Highway work necessarily requires the maximum of time in the spring and summer, leaving but little work for winter. For that reason some of the colleges have arranged short winter courses for highway superintendents; and while during their stay at these institutions the men cannot cover with great thoroughness all of the matters which they should understand, the courses nevertheless stimulate them to continue their studies and reading after returning home. Some of the state highway departments hold conventions and meetings with the same object in view.

Attention is called to this matter now because the arrangement of such a course is not a matter of a few days. It requires study of the needs of the particular state or locality, conference with competent highway engineers, and investigation as to what other colleges have done and how they have fared. All of these things take time and unless the matters are taken in hand before the student body begins to return late in September there may be but little opportunity for busy deans and professors to give the matter adequate consideration. The course, it is true, would probably not be given until after the holidays, but ample preparation, a suitable program and an educational campaign among highway officials are necessary for satisfactory results.

Structural Engineering Notation

A comprehensive addition to the notation resources of modern structural engineering was presented at a recent meeting of the Concrete Institute in London by the Science Standing Committee of the organization, following an extended investigation of the possibilities of establishing mathematical notation on the index principle. The importance of using logically selected symbols is more and more evident as the work of the engineer extends beyond local areas, regardless of the special branches of the profession practised here or there. The interchange of technical thought between the world's antipodes is one of the first requisites of broad progress and the use of a well-standardized notation clears away many barriers to the speedy comprehension of the ideas of others. While it is true that the establishment of a system of symbols which shall become universally accepted and above all employed in practical calculations is an ideal which may never be fully realized, the compilation of logical notation in different engineering branches and its wide distribution among specialists are important steps forward, for they provide labor and time-saving tools for common use and contribute to the community of interest throughout the profession.

The report of the committee sets forth the principles by which symbols may be selected, tabulates the kind of type most suitable for different kinds of quantities, and presents detailed lists of principal algebraic symbols used in statical calculations for all branches of structural engineering, including reinforced concrete, steel, timber, brick and masonry work. The entire notation is built on the principle of an alphabetical index, and the committee has endeavored to set forth a system giving a certain freedom of choice on the part of the user while still retaining the maximum of consistency and the fullest development of the mnemonic principle. So far as practicable the symbol selected is the first letter of the word indicated, various tensions, for example, being expressed by the letter "T," either in lower-case type, italics, capitals, Greek or Roman forms, by subscripts, or subdashes. Similarly, all bending moments are expressed by some form of the letter "B," inertia moments by some form of the letter "I," and so on. In the selection of Greek letter symbols for angles with the horizontal or with the vertical the approximate direction of the mnemonic straight-line in the symbols θ and φ is emphasized. The system also provides for expression of increasing or diminishing quantities, listing of quantities not subject to the initial letter principle, notation of constants individually and in series, grouping of consecutive series of points or spaces, products of lineal dimensions, ratios, coefficients, moduli, comparison of totalities, vectors, angles, dimensions, degrees and other considerations with which the designer and the testing and operating engineer have to deal. Nearly one hundred symbols are proposed for general use in all branches of structural engineering; those recommended for adoption by reinforced-concrete specialists are still more numerous, and the other branches listed are by no means scantily supplied.

The system is illustrated in the report by the use of English units, but is equally adapted to metric or other units. A careful examination discloses an unusually small number of exceptions to the rules given. Thus, in dealing with vectors, since vectors are symbolized by Greek lower-case letters, any necessary angles are represented by italic capitals in seriate order. Again, superscript symbols indicating powers, roots and units employed are necessarily used with special care, those indicating units being employed only in a few cases where two different quantities, occurring in the same set of equations, have the same initial letter, but are in different units. Terms in equations which have natural associations are required to be grouped together, and in standardizing forms for equations the report emphasizes the importance of preceding variables by constants, conserving the customary order of terms, and arranging symbols in final equations to fall in alphabetical order so far as is reasonably practicable.

It is clear that in using notation for general circulation a key should always accompany the data and calculations, and the committee has acted wisely in recognizing this point. There is a natural temptation to the compiler or creator of any logical and consistent system of symbols to assume that such a system is self-sufficient. Close familiarity with notation tends to make the user overlook the point that quantities and constants which he continually employs may be utilized only rarely by others in their particular problems. For this reason even the more common symbols employed in scientific treatises are practically always set forth in key form by authors who are determined to leave no opportunity for misunderstanding. In the electrical field, for example, the letter "R" is universally recognized as the symbol for resistance, but in careful works, where equations are being derived, the exact meaning of even this cornerstone of the structure of electrical measurements is properly set forth in the key. The desirability of a key in nowise invalidates the usefulness of a system of notation, and the ease with which such methods of what might be called mathematical shorthand can be applied by beginner and expert alike commends them to the test of actual service.

Putting Life into the Meetings of the American Society of Civil Engineers

It is well known that the meetings of the New York branch of the American Society of Civil Engineers are very dull. The attendance is small and there is so little comment on the papers that it is farcical to call it "discussion." Furthermore, there is a stiffness and lack of cordiality that is not dispelled even by the very satisfactory refreshments served after the technical ordeal. Sporadic efforts have been made to better the conditions, but without result. The board of direction recently had the matter under consideration and has formulated new regulations for the presentation of papers which are likely to result in some improvement. Whether they will bring about the same spirit that animates the meetings of the San Francisco and Denver associations of members is doubtful, because unfortunately the New York meetings are regarded as meetings of the society as a whole, instead of local gatherings.

Since these new rules, printed on page 78 of the Engineering Record of July 19, are likely to be copied by other organizations it may be well to consider some of them in detail.

The determination to have the authors present papers in person or to designate someone to read them is a decided improvement over the present method of reading by title, which has degenerated into a standing joke. It would be better, however, to rule that unless the author agrees to present the paper in person or to secure the services of someone conversant with the subject, in sympathy with his views and deemed competent by the committee on publications, the paper shall not be presented but merely printed in the "Proceedings." Unless a competent person is prepared to defend the author's viewpoint the paper is certain to fall flat—the experience of 90 per cent of those now presented.

While the scheme of having the author supply a list of members to be invited to discuss the paper has been used with success in other societies, the invitation should be supplemented by a crisp summary not over three hundred words long, for such a summary would interest many who would not be impressed by the title of the paper. This summary should be shorter than the digest to be read at the meeting.

The new rules lead one to infer that all papers which lend themselves well to discussion will, as in the past, be read at New York. If this is the case, it is a distinct error, for there are many papers, most valuable in themselves, which cannot be discussed intelligently at New York because few engineers there are sufficiently conversant with the subject. Papers on irrigation, for example, are invariably read by title and elicit little or no comment, whereas the same papers read in Denver and San Francisco undoubtedly cause valuable discussion. The problem the board of direction has started out to solve is not the best method of presenting valuable papers but of making the New York meetings interesting and worthy of the dignity of the society. If such is the case, the standard must not be the intrinsic merit of the paper but the extent to which it will arouse local interest.

The regulation making the printing of his discussion optional with the member is likely to be one of the most effective in stimulating discussion. Heretofore he has felt that he was speaking not to an intimate group of local members but to the society at large and free discussion has accordingly been stifled. He hesitated to go on record before the profession as asking simple questions or airing views that might arouse adverse criticism. The rule should tend to dispel much of the stiffness and formality and make the meetings as they should be—free forums for the exchange of ideas.

While the new rules, therefore, should have a decidedly favorable influence upon the New York meetings, they can, with benefit, be still further amended. It is well worth considering, too, whether there should not be some recognition of the fact that these meetings, composed, as they are, of residents of the metropolitan district, are local and not national. While there is some merit in the view that the society should have formal meetings, it would seem to be far better to develop a spirit of fellowship and co-operation among the New York members than to continue the present artificial basis upon which the semi-monthly meetings are held. Until local subjects are occasionally introduced, and the members thereby brought more closely together, the engineers of the metropolis will certainly not reap the benefit that is secured by those who attend the meetings of active branches. That local subjects will completely change the character of a meeting was clearly demonstrated some time ago when Mr. Wiggin, designing engineer of the New York Board of Water Supply, described in an informal manner some of the novel shaft work on the Catskill aqueduct. Moreover, until the New York members are better organized they will fall far short of doing their civic duty in the way that smaller engineering societies are doing it. That there is need for such co-operation has been apparent a number of times, particularly when the New York building code was up for consideration some years ago and when the first licensing bill was proposed in New York State.

The strength of the society consists not only in maintaining its present dignity but also in developing an active, interested, responsive membership. The metropolitan district should be given advantages similar to those enjoyed by the existing branch associations. Their form of organization and method of conducting meetings, particularly at Denver and San Francisco, have been decidedly successful, and it is hoped that the introduction of the new rules and their modification along the lines here suggested will put life into the meetings at New York.

Construction Camp at Arrowrock Dam

How Necessities and Amusements Are Provided for 1400 Workers Located 20 Miles from Boise, the Nearest City

By Alfred B. Mayhew, Engineer, U. S. Reclamation Service

The construction camp established by the U. S. Reclamation Service at the Arrowrock dam in Idaho embodies advanced ideas in camp construction and maintenance. Among the more notable features may be mentioned a water-supply system, a sewerage system, electric lights, a central heating plant, a clubroom, a post office, a public bath house and a laundry or wash house, where the laborers may wash and dry their own clothes. Special attention has been paid to sanitation and to comfortable housing and recreation of the men.

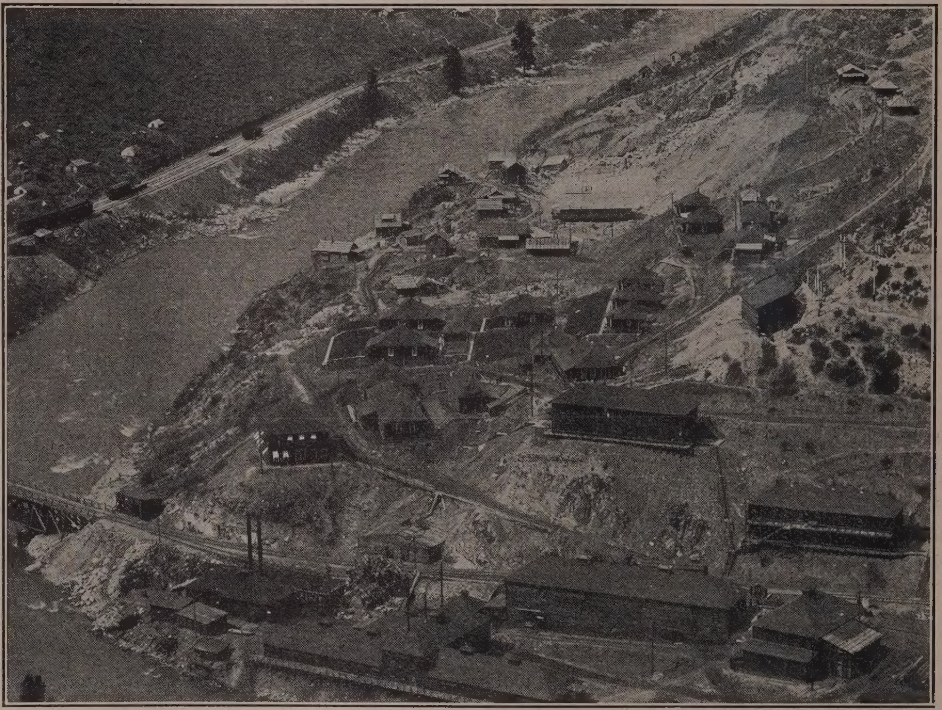
Arrowrock is located on the Boise River, about 20 miles in an easterly direction from Boise, Idaho. In the vicinity of the camp and dam site the Boise River flows in a wide canyon, as may be seen from the illustrations. The mountain sides are of granite rock and granite sand, practically devoid of all vegetation except the sage brush and a few scattered pine trees. There are no cities or towns nearer than Boise, and for that reason an effort was made to have the camp as attractive and homelike as possible.

The camp buildings are located on the north bank of the river, about one-quarter of a mile below the dam site. This location was the only one that was in any way desirable for a camp site, and even here the space that could be utilized to advantage was small and the topography such that a large amount of grading was required. In several places the granite bed rock extends to the surface, and for that reason it was impossible to locate the camp buildings along regular lines without incurring unreasonably great expense. All construction in connection with the camp is of a comparatively permanent nature, as it will require about five years in which to build the dam and related features. The total population of the camp is about 1400 and includes about 200 families. A school has been organized which is attended by about 90 pupils and requires the services of two teachers. The subforemen, mechanics and skilled laborers were encouraged to establish homes in the vicinity of the dam site, and at present there are more than 175 of these men, with their families, living in comfortable little cottages. These little settlements, however, are not considered as a part of the main camp, as none of the construction was done by the Reclamation Service; but as nearly all of these cottages are on public lands the camp authorities exercise supervision over them and enforce the same sanitary regulations as are required in the main camp. The main camp was constructed during the summer of 1911, and was designed to furnish adequate office and shop facilities for properly conducting the construction work, together with satisfactory housing for about 750 single men and 18 families and dining room capacity for about 600 men per meal.

Description of Cottages

The main camp consists of the following buildings: Fourteen cottages, mess house for the engineers and office force, mess house for skilled and common laborers, office building, warehouse with cold storage and ice plant attached, general store and clubroom, post office, general hospital, isolation hospital, four dormitories for engineers and skilled laborers, bakery, central heating plant with public bath house attached, laundry, stable, general workshop, locomotive engine shed, electric shop, cement-testing laboratory, plumbing shop and other small or unimportant buildings.

Twelve of the fourteen cottages accommodate one family each, and the two remaining cottages are arranged for two families each. The single cottages range from 28 x 30-ft. buildings, containing four rooms and bath, to 18 x 29-ft. buildings, containing three rooms and bath. Each single cottage and each apartment of the double houses is furnished with screened porches, both front and rear, and with a porcelain bathtub and toilet. All of the cottages are plastered throughout, and all inside and outside woodwork is stained. The roof covering is of one thickness of Cronolite roofing laid on 1-in. sheathing. The floors consist of tongue-and-groove flooring laid on 1-in. sheathing. A special feature of the cottage construction is the manner of laying the outside sheathing, which consists of 1 x 12-in. boards lapped 10 in. to the weather. Each board was nailed to the stud so that the nail just cleared the top of the board just below, the top of each board being held in place by the overlap of the board next above. Thus there was no hindrance to shrinkage, and shrinkage cracks were obviated. Later, after the lumber had ceased shrinking, the overlapping portion of each board was nailed through the board lying beneath it and into the stud. This method of construction was necessary because all the lumber used was in a comparatively green condition. The cottages are rented at prices ranging from $10 to $16 per month, which, in course of five years, will practically repay the original cost of each cottage.



Cottage Section of Construction Camp at Arrowrock Dam

The dormitories range from one-story to two-story buildings, and differ somewhat in construction, but a description of one will serve to illustrate their general arrangement and construction. The mechanics' dormitory is a 25 x 80-ft. two-story building, having thirteen sleeping rooms and a bath on each floor. The sides and roof are of 1-in. sheathing, covered with Neponset building paper and Cronolite roofing respectively. All inside partitions are covered with building paper, and the floors are of double pine boards. Each room is made to accommodate two men, and contains two cots, each supplied with springs and mattress. A charge of $2 per month per man is made for the use of these rooms, and includes light, heat and janitor service. Many of the men take great pride in their rooms and have purchased chairs, desks and curtains, and have decorated the walls with pictures and flags.

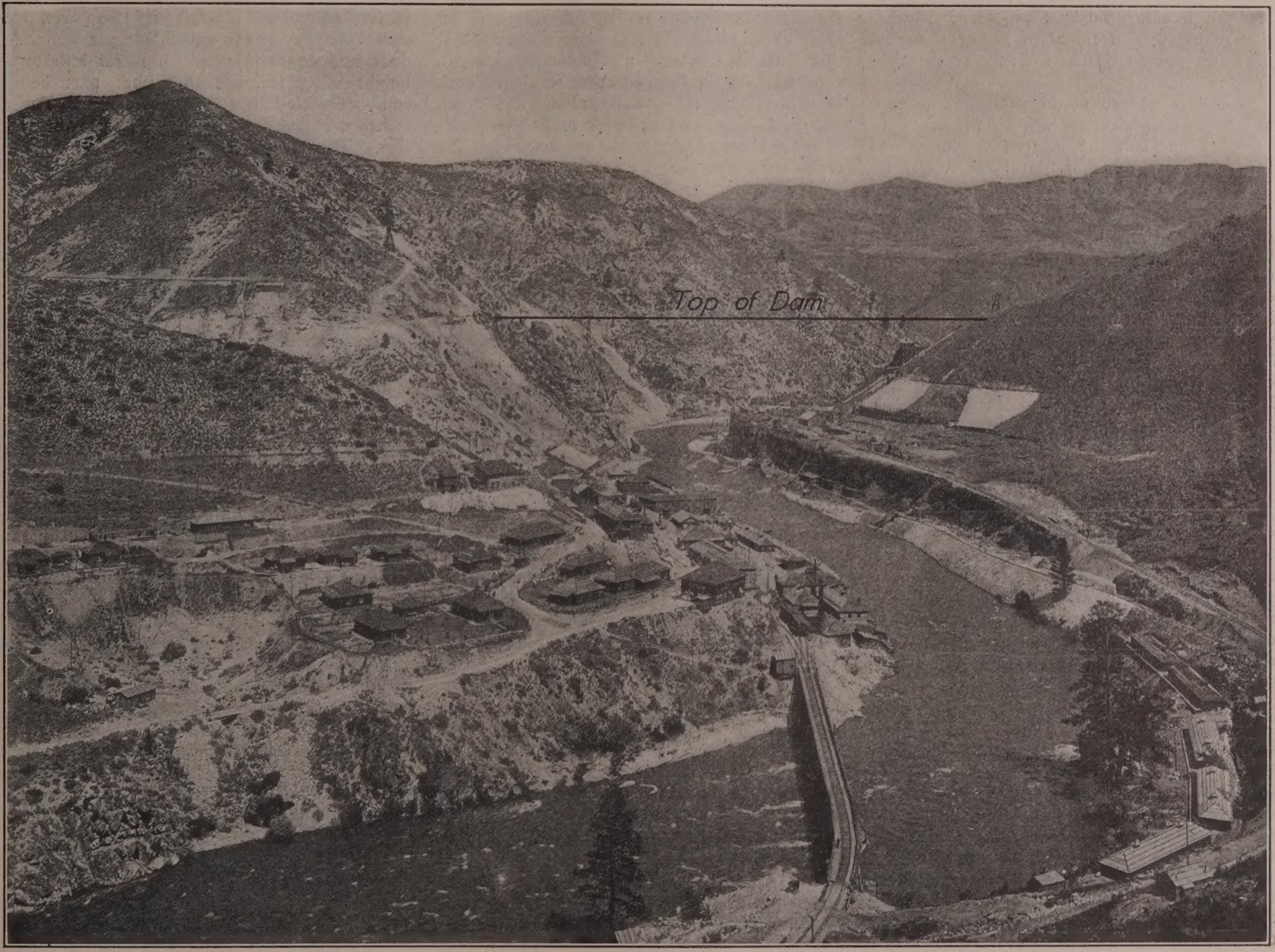
Eight of the ten bunk houses are 18 x 91-ft. buildings, each consisting of two bunk rooms and accommodating 56 persons. The bunk rooms in each building are separated by an 11 x 18-ft. washroom, containing two large sinks. Each room is also provided with a 13½ x 18-ft. general sitting room, furnished with benches and tables. The bunks are 2½ x 7 ft. and are built two rows high. Each bunk is supplied with a good set of springs and a mattress. No charge is made for the use of these bunks. The construction of the bunk houses is of the same general type as that used in the dormitories.

Mess House

The mess house for the skilled and common laborers will accommodate about 600 persons at one time, and consists of a kitchen, 25 x 66 ft. in plan and 18 ft. high to the eaves; one dining room, 30 x 40 ft., and two dining rooms, each 30 x 75 ft. When a man is employed he is given a slip, which he turns in to the steward before receiving his first meal. As nearly all the men have regular places at the tables it is easy for the waiters to observe any new men and ascertain if they are bona-fide employees. The steward is required to keep a record of all meals served, and this record is checked against the time books daily.

A small dining room is provided at one end of one of the large dining rooms for the convenience of visitors and transients. The patrons of this dining room are required to pay cash for all meals served. The kitchen is provided with a concrete floor, and the dining rooms are provided with board floors. The sides and roof of the entire building consist of 1-in. sheathing covered with Cronolite roofing on the outside and are whitewashed on the inside. Large ventilators are provided in the kitchen and in each of the dining rooms. All the entrance doors are screened and are also provided with screened porches. The kitchen is equipped with a Schilling center range, having a cooking surface of 84 sq. ft.; two steaming and carving tables, each 12 ft. long; a Blakeslee vegetable and potato peeler run by a 1-hp motor, a Blakeslee vegetable creamer run by a ¾-hp motor; a four-compartment steam vegetable cooker, and a Blakeslee No. 6 dish washer run by a 1-hp motor. The tables in the dining rooms are 4½ x 5 ft., and each accommodates six to eight persons. This size of table seems to be popular with both the men and the management. With only six or eight persons at a table a man is able to find a place with his particular friends and is not obliged to eat with men who are uncongenial. It has also been found that with the small tables there is less loss on account of food getting cold than with the long tables usually used in camp dining rooms.

The warehouse, cold storage and ice plant are located in a 30 x 125-ft. building, consisting of one story and basement. The ground floor is used for storage of hardware, and also includes the warehouse office and the ticket and freight offices of the Arrowrock railroad. The basement is divided into two distinct parts; one part, 68 ft. in length, is used for storing groceries, etc., for use in the store, and the other part, 52 ft. long, is used as a meat market, refrigerating plant and ice plant. The main refrigerating room is 20 x 30 ft. in plan and has a capacity of about 26,000 lb. of meat. There is also a provision room 10 x 13½ ft. The ice plant is equipped with a 6-ton ammonia compressor operated by a 10-hp motor and with a 1½-in. centrifugal pump operated by a 2-hp motor. The capacity of the ice plant is 2000 lb. of ice per twenty-four hours, in addition to serving the refrigerating plant. The meat market is equipped with a meat grinder and a coffee grinder, each operated by a 1-hp motor.



Site of the Arrowrock Dam in Idaho and General View of Construction Camp

Store and Clubrooms

The store and clubrooms occupy a building consisting of one story and basement. The basement, 36 x 62 ft. in plan, is used for the store, and the ground floor, 36 x 50 ft. in plan, is used for the clubrooms. The store carries a very complete stock of general merchandise, including groceries, boots, shoes, ready-made clothing, toilet articles and sundries of all kinds. There is also a soda fountain and an ice-cream room. As far as possible, prices are maintained at the level of prevailing prices in Boise. The store, meat market and mess houses are not maintained as money-making schemes, yet they yield a small profit.

The clubrooms consist of a general reading room, a writing room, a sleeping room for the attendant and a general office. The reading room is provided with nearly all of the leading newspapers and magazines, and books are supplied by the Idaho Free Traveling Library. This room is open from 8 a. m. to 10 p. m. and is free to all employees. There is also a piano, a phonograph with a good set of records and a few games, the use of which is free; but a small charge is made for the use of the two pool tables. On two nights a week a motion-picture show is conducted, for which the regular charge is made. A creditable orchestra has been organized from among the employees, which furnishes music for the picture shows and other entertainments. The field division of the Y. M. C. A. co-operates with the Service in conducting the club, and the piano, motion-picture machine, pool tables, newspapers and magazines are furnished by that organization.

General Hospital

The general hospital is a 24 x 48-ft. building with a wide screened porch around two sides. It contains a waiting room, office, operating room, ward, bathroom and three private rooms. The construction is of the same general type as that used in the cottages, except that the plaster has a trowel finish and is coated with white enamel paint, as is also all of the exposed woodwork. A twocompartment Copeman automatic electric stove and an ice chest are used to good advantage, although meals for the patients are regularly prepared at the office mess.

When a man is employed he receives an identification slip, which he must present at the hospital, where he is given an examination by the attending physician. The applicant's physical condition is noted on the identification slip by the physician, and, if satisfactory, the applicant is given a number and regularly enrolled on the time books upon the return of the slip to the general office. A charge of $1 per month for hospital dues is made against each employee, which guarantees the necessary medical and surgical attention in cases of accident or sickness, except disability due to causes existing at the time of employment.

The rules governing the procedure in case of accident or sickness are as follows: In cases of accident the injured employee will report at once to the hospital for treatment. Meals will be furnished without charge, provided the accident was not due to negligence or misconduct, until such time as hospital service is no longer needed, but not to exceed one year. In cases of sickness causing incapacity for work the employee will report immediately to the physician; if it becomes necessary to make him a bed patient in the hospital meals will be furnished without charge from the time he is admitted to the hospital, or in any case after seven days from the time he first reports to the physician until medical attention is no longer needed, but not to exceed three months. Meals will not be furnished free of charge to employees who are incapacitated for work on account of chronic or venereal diseases, or disability due to causes existing at the time of employment, or minor ailments that terminate during a period of seven days or less; nor will free meals be furnished for the first seven days of sickness, except in cases where the patient is necessarily confined to the hospital.

CENTRAL HEATING PLANT

The central heating plant and the public bath house are under one roof. The walls and ceilings of the bath house are of cement plaster over metal lath, and the floors are of concrete throughout. The heating plant contains two Kewanee 50-hp high-pressure boilers, which are used for heating all of the camp buildings except the cottages. These boilers will also be used to furnish power for the emergency fire and water supply pump, in case the operation of this pump becomes necessary. The bath house, 24 x 26 ft. in plan, contains two tub baths, six shower baths and a dressing room. An addition has been built on the bath house, in which there is a barber shop equipped with three barber chairs.

The laundry is a 12 x 30-ft. building, containing six stationary tubs and several drying racks. This house was built for the convenience of the laborers who do their own washing. The tubs are supplied with hot and cold water, and the drying racks are heated by steam pipes from the heating plant.

The general workshop is 30 x 25 ft. in plan and contains a carpenter shop, a machine shop, a blacksmith shop and a tool room and office. The carpenter shop is equipped with a 12-in. Universal woodworking machine and an 18-in. planer. The machine shop is equipped with lathes, drill presses, sharper, splitting shears, air drills, wheel press, hand punch and shears, a bolt machine and other standard tools sufficient for all ordinary repairs and for the construction of much of the equipment and many of the tools required on the work. The blacksmith shop is equipped with three concrete forges served by a Buffalo blower, a power hammer, a bolt machine, a tire shrinker, a tire bender, a drill press and other standard tools.

Nearly all of the lumber used in the construction of the camp was hauled by teams from a sawmill installed by the Service on a tributary of the Boise River, about 18 miles above Arrowrock.

WATER SUPPLY

The entire camp is supplied with an excellent quality of water obtained from Deer Creek a small tributary of the Boise River. An earth dam about 15 ft. high was built on Deer Creek about 1¼ miles above Arrowrock, thus forming a reservoir of about 800,000 gal. capacity. As the land adjacent to Deer Creek is public land, all grazing along this creek was prohibited, and in order to further safeguard the water supply the reservoir and the creek for a distance of 1 mile above the reservoir was securely fenced. From the reservoir the water is brought along the mountain side in a 12 x 12-in. box flume embedded in the ground. At the north end of the dam site this flume empties into a 35,000-gal. tank about 70 ft. above the crest of the dam and about 310 ft. above the lowest portions of camp. This elevation not only enables all parts of the camp to be furnished with a gravity supply of water for domestic purposes, but also furnishes an excellent high-pressure supply for fire protection at the camp, and makes the supply available for construction purposes at the dam. From the storage tank a 4-in. iron pipe conveys the water to the dam site at about the elevation of the top of the dam, and a 6-in. wood-stave pipe conducts the water through the entire camp on the north side of the river, whence it is distributed through pressure reducers to the service pipes leading to the various buildings. A 3-in. iron pipe, carried on the railroad bridge, serves the buildings on the south side of the river, and a 4-in. wood-stave pipe takes off from the 6-in. main just as it enters camp and is carried to the dam site at about river level. A 10 x 2 x 6-in. Fairbanks-Morse duplex pump has been installed near the heating plant. This pump will be used to supply water to the camp mains from a well located on the bank of the Boise River, in case the supply from Deer Creek should become inadequate during the summer months, or in case of injury to the main between the supply tank and camp. This pump is also provided to secure an instantly available additional supply of water in case of a disastrous fire.

SANITATION

Sanitary conveniences have been provided throughout the camp and on the works, and all persons connected with the works are required to use these conveniences. Tight, galvanized iron garbage cans have been provided at each of the camp buildings and at convenient points along Deer Creek and at other points among the private cottages. Every day in summer and three or four times a week in winter the contents of these cans are hauled away and burned each day, and no rubbish of any kind is allowed to accumulate. During the summer of 1911 a number of large fly traps were placed at different places throughout the camp, but in 1912 there were so few flies that only a few of these traps were used. The hospital physician is at the head of the sanitary department and is required frequently to inspect the dwellings, stable and sanitaries. A camp boss, with a corps of watchmen and janitors, is provided to maintain the camp and camp buildings in a most sanitary condition and to keep order.

SEWERAGE SYSTEM

A concrete septic tank of about 25,000 gal. capacity was built near the river, and all camp buildings in the camp proper are connected with this tank by means of wood-box sewers varying in size from 5 x 5 to 10 x 12 in., inside dimensions. These sewers are flushed occasionally from hydrants and have given no trouble whatever. The bunk houses are not provided with toilets, so five public toilets were built near these buildings. The public toilets on the north side of the river are 7 x 16 ft. in plan and are divided into two parts, one part being a urinal and having a concrete floor and concrete sides on metal lath to a height of 3 ft., and the other part being equipped with four frost-proof closets. These toilets are also connected with the sewer system. The toilets on the south side of the river are dry closets.

FIRE PROTECTION

Six two-way fire hydrants, connected directly to the 6-in. camp main, and several 2½-in. hose connections from the 4-in. wood main and the 3-in. iron main have been installed and command the entire camp. Each hydrant and hose connection is supplied with 150 ft. of 2⅛-in. linen hose with a smooth 1⅛-in. nozzle, all stored in a small hose house and connected with the hydrant ready for instant use. In addition to this there is a hose cart with 800 ft. of hose stored in a hose house near the office. Each cottage is supplied with lawn connections and ¾-in. lawn hose, which, with the pressure available, would be sufficient to put out a small fire in any part of the house. The office, warehouse, cement plant, store and clubrooms are supplied with 5-gal. Babcock chemical fire extinguishers. A fire bell, which can be rung from the telephone booth, where some one is on duty at all hours, has been placed near the general-office building. There is also a fire whistle provided at the central heating plant, and a system of fire whistles has been established. Fire ladders and axes are provided at convenient points throughout camp, and a volunteer fire department, consisting of two companies of twenty members each, has been organized.

AMUSEMENTS

In addition to the games, pool tables and picture shows provided at the club there have been illustrated lectures given by members of the Service, and many entertainments provided by local talent. Among the private buildings there is a hall, 30 x 60 ft. in plan, in which public dances are held each week, and during the winter season this hall is also fitted up for basketball. There are also a baseball diamond and a good tennis court, both of which have been prepared by the employees. Gambling and excessive drinking are prohibited, and no liquor is allowed to be sold in the vicinity of the camp. So far there has been very little trouble from "bootlegging."

All work in connection with the construction of the Arrowrock dam is being done by government forces under the direct supervision of Mr. Charles H. Paul, construction engineer, who reports to Mr. F. E. Weymouth, supervising engineer of the Idaho District of the U. S. Reclamation Service. Mr. James Munn, superintendent of construction, is in direct charge of all construction work. The writer is principal engineering assistant to the construction engineer.

Tests of Pipe Joints

By A. J. Cleary, Assistant Engineer, Board of Public Works, San Francisco

In connection with an article describing the San Francisco auxiliary water supply system for fire protection which was published in the Engineering Record of July 26, 1913, page 107, there was a reference to tests conducted by the engineering department of the city to determine a type of bell and spigot and spigot sleeve cast-iron pipe joints that would most adequately withstand any lateral movement without destroying the permanency of the joint. These tests were made at the plant of the Union Iron Works, of San Francisco. The cross-section of each type of bell joint tested is shown in Fig. 1, while the results of the tests are shown in Figs. 2, 3, 4 and 5. In making the tests a bell and plug such as shown in Fig. 1, No. 1A, were used. The bell was firmly clamped in a vise; the plug was fitted into it, the joint yarned for 1 in., and the remaining 3½ in. poured full of lead. The joint was then thoroughly calked with 3 sets of calking tools, having faces 1 in. long and 3-16 in., 5-16 in. and 7-16 in. wide respectively. Through a ⅜-in. pipe tap water was introduced and pressure applied by means of a force pump. The amount of leakage per minute was measured for various pressures, and also the distance the plug moved out of the bell. Fig. 2 shows the final pressure developed for each type of joint just before the plug blew out; also, the pressure per square inch applied before leakage began. Fig. 3 shows the distance the plug was forced out of the bell by various pressures. Fig. 4 gives graphically the relation between bending moments and the deflection produced by them at the end of a 12-ft. length of pipe, provided with the three types of bell joints indicated. The relation of leakage to pressure, for bells of the type indicated, is given in Fig. 5.

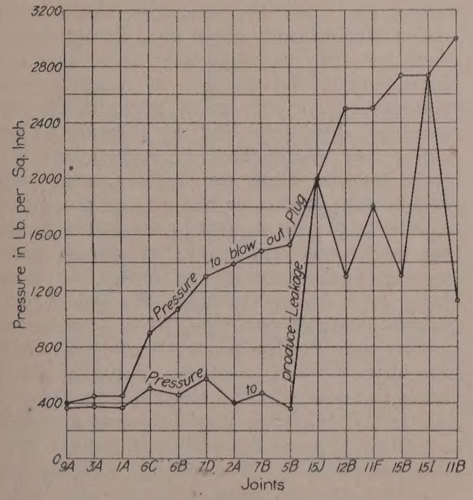


Fig. 2—Unit Pressure at Failure

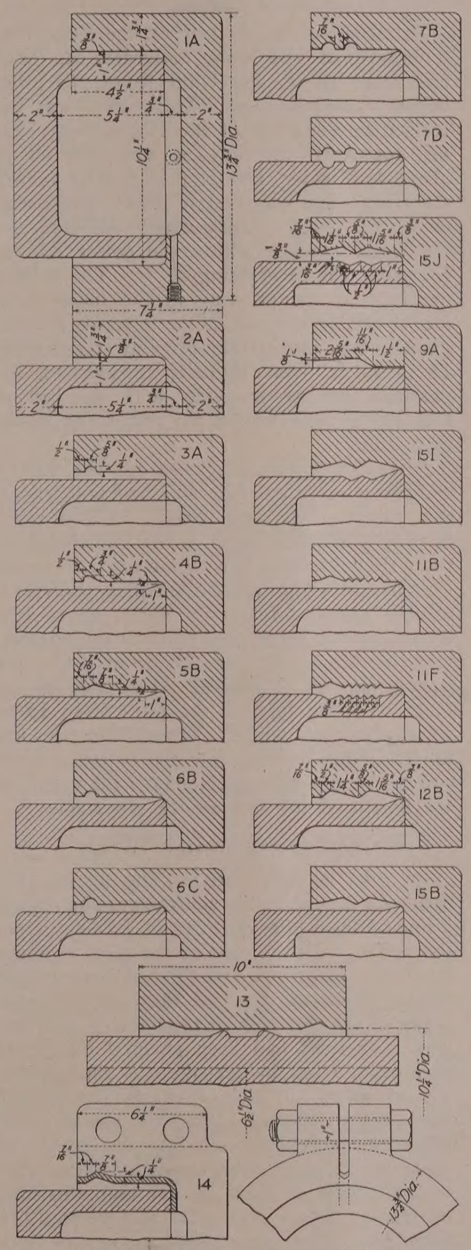


Fig. 1—Testing Rig and Types of Bell Joints Tested

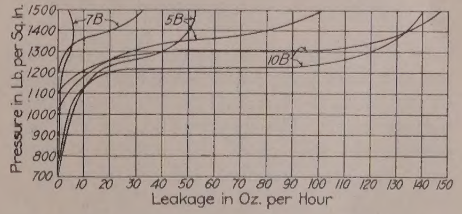


Fig. 5—Joint Leakage in Tests

Thermophones for Temperature Measurement in Dam

Tests to determine the thermal changes in large masses of masonry are to be undertaken by the Board of Water Supply of the City of New York at the Kensico dam, Valhalla, N. Y. This dam will form a large storage reservoir on the line of the new Catskill aqueduct, and is being built of cyclopean masonry, faced on the upstream side with molded concrete blocks and on the downstream side with cut granite. The Kensico dam is to be provided with vertical expansion joints, and the studies of the effect of temperature changes of the mass of masonry and the internal stresses set up, the engineers believe, will be of peculiar interest. The information is to be secured by means of thermophones embedded in the masonry during the construction of the dam.

Grouping

The thermophones will be installed in two groups. Those in one group will be placed at various distances from the face in a vertical plane midway between expansion joints. The second group will consist of a similar arrangement near the face of an expansion joint to determine the effect upon the temperature of carrying up the masonry on one side of the joint in advance of the other.

Coils will be placed in the refill near the upstream and downstream faces and also in holes drilled in the foundation to determine the temperature of the bed rock. Each thermophone consists essentially of two coils of many turns of fine wire of two metals having different temperature coefficients of electrical resistance, connected in circuit with batteries and a galvanometer. The wires are of such size and length that a small change in temperature causes a measurable change in electric resistance, which is indicated by the gal-

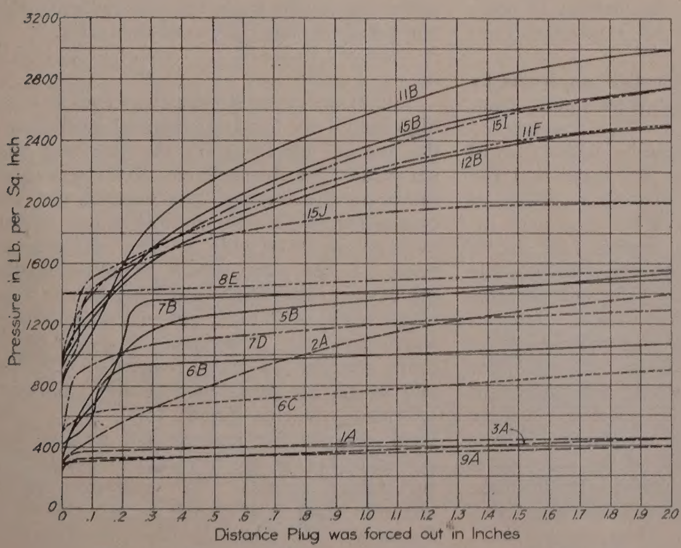


Fig. 3—Outward Movement of Plug under Pressure

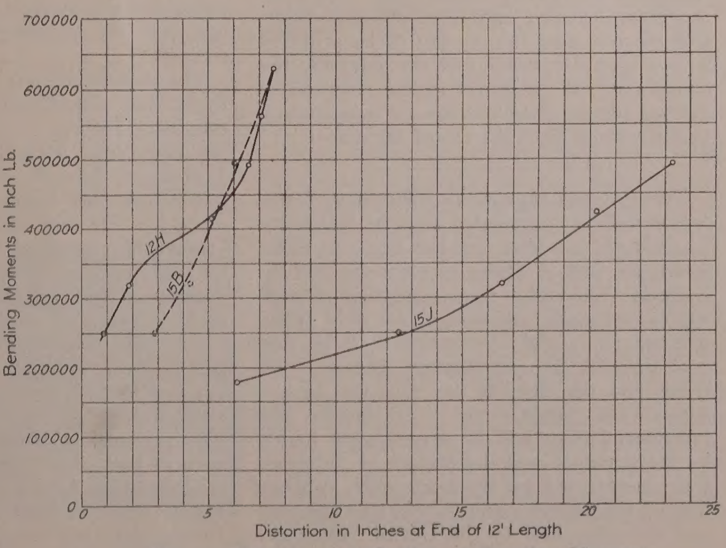


Fig. 4—Relation of Deflection to Bending Moment

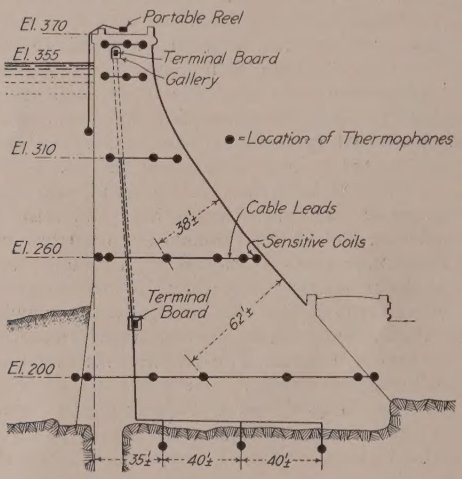
vanometer, or detected by a telephone by moving the pointer of a Wheatstone bridge until the silent point is reached.

The sensitive coils are constructed of copper and German silver wire, and enclosed in a brass sheath about ½ in. in diameter and 8 in. long. To the sensitive coil is securely soldered a cable consisting of insulated No. 16 copper wire protected by a lead covering about 1/16 in. thick. The cable weighs approximately ½ lb. per foot.

Equipment

The equipment consists of sensitive coils embedded in the masonry and connected by cables leading through the masonry to terminal boards located at convenient places where indicating instruments may be attached and observations taken. The terminal boards will be located in the upper and lower inspection galleries. The cables will be either coiled in boxes or placed upon reels in such a way as to afford access to the terminal end for attaching the indicating instrument, so that readings may be made as soon as the sensitive coils are embedded in the masonry.

The cable will be unreeled as fast as necessary for placing in the masonry, and special care will be taken to protect the boxes until the work has progressed far enough to permit of installing the terminal boards in the inspection galleries. The terminal boards will be so constructed that after the indicating instrument has been attached, observations may be made on any circuit by the use of a dial switch.



Location of Thermophones in Dam

Two indicating instruments will be used, one calibrated between 32 and 100 deg. Fahr. and the other between zero and 180 deg. Fahr. One thermophone is proposed to be provided with a special portable reel for obtaining the temperature of the water in the reservoir at various depths, as well as the air temperature, by lowering it from the top of the dam.

The Palouse Irrigation Project in Washington will be investigated as to its feasibility. To this effect a contract has been made between the United States Government and the State of Washington for co-operation in making surveys and estimates. All field notes, original plans and other data shall be filed with the U. S. Reclamation Service. Of these data the state at any time can have a copy upon application. Conclusions and recommendations will be agreed upon jointly by the governor and the officers of the Service. In carrying out the work under contract half of the expenses will be borne by the state and the other half by the United States, the total for each not to exceed $10,000.

Failure of Cofferdam at Ohio River Lock and Dam 48

By Major John C. Oakes, M. Am. Soc. C. E., Corps of Engineers, U. S. A., Louisville, Ky.

At about 7 a. m., on July 21, there occurred a failure of the main cofferdam at Lock and Dam 48 on the Ohio River, 18 miles below Evansville, Ind. This coffer was described on page 412 of the Engineering Record for April 12, 1913; it was built 20 ft. above low water, and at the time of failure the river stage was 12.4 ft. above low water, or at El. 337.5 (Sandy Hook datum), and the water surface within the coffer was at El. 316.5, the head being 21 ft. The failure occurred at the passway in the lower arm, left to enable the contractor to move floating plant in and out of the coffer at stages from 18 to 20 ft. As originally constructed, the passway was 41 ft. wide, with the bottom 6 ft. below top of coffer, the area being closed by needles.

After the work had closed for the winter and the government inspector had withdrawn, this passway was torn out to remove some floating plant, and without the knowledge of the writer was rebuilt with the top 2 ft. lower than before. The top of the coffer, therefore, at this passway was only 12 ft. above low water, needles being used to close the opening when required. On the outside of the coffer was a line of Wakefield sheet piles 26 ft. long, with the tops of the piles from 10 to 12 ft. above low water. These sheet piles extended about 10 ft. below the bottom of the coffer. Sand was banked against the coffer to the height of the tops of the sheet piles.

Precautions Against Wash

In order to hold the banking against the coffer on the inside the contractors had driven a line of sheet piles 60 ft. or 65 ft. away from the coffer, with the tops of the piles at about low-water elevation, and fill had been placed sloping from the tops of these piles to the coffer at the elevation of the floor of the passway. This fill had been covered by gravel to prevent wash by seepage. A small quantity of water, probably 1 cu. ft. per second, had always collected behind the passway and flowed as a little stream into the excavated area, but as there were many of these little streams, some of them greater than this one, no fear of failure because of this seepage had been felt. The government inspectors lived on a quarterboat, moored just below this passway, and crossed the coffer constantly at this point, and no increase in the amount of seepage nor any movement of sand had been noticed.

The excavation was kept clear of water by three 15-in. centrifugal pumps on a pump boat, the latter resting on piles. The engineer of the pump boat noticed nothing unusual until 5:30 a. m., July 21, when he found that the pumps were not holding the water surface at El. 315.5. He then changed his governors, increasing the speed of the engines, and finally cut out the governors entirely, allowing the engines to run full speed. In spite of this the water surface inside rose to El. 316.5 at 6:30 a. m. Shortly after this the men coming to work noticed a large leak near the passway, the alarm was given, and the pumps stopped at 6:45 a. m. At 7:01 a. m. there was evidently a blowout, as the water was seen to suck down outside of the sheet piles, and the sheet piles were lifted out; then the coffer lifted and the break was completed. Before the inclosure was filled about 250 ft. of coffer had been washed away.

Through the gap thus created there were drawn four loaded coal barges, a barge of lumber, and one of round piles. The coal barges were rolled over and over, and were a total wreck; the pile barge was broken up, and the lumber barge was injured, but can be recovered and repaired. The pump boat was thrown off its pile foundation and somewhat injured, and four pile drivers were submerged and probably injured to some extent, the amount of injury being unknown.

The contractor was about ready to place concrete for the river wall. The excavation was finished, all of the round piles and most of the sheet piles were driven, and tracks on piles for derricks and cars were completed. A large amount of sand has been carried into the excavation, coal has been dumped about the heads of the piles, and undoubtedly some of the tracks have been injured. It is estimated that the immediate money loss to the contractor will amount to between $10,000 and $15,000, but the accident may cause a much greater loss due to the delay, which is estimated at a month or six weeks, which may prevent the completion of the work inside the coffer this season and make necessary the unwatering again next year. It is hoped, however, that further investigation when the river falls will show less damage than is anticipated.

Causes for Failure

Several causes for the failure can be suggested, and it is probable that they all had a bearing; their relative importance, however, can only be guessed at. These probable causes are: The weight of the passway coffer was probably about 1000 lb. per square foot less than it would have been if it had been built to full height; in rebuilding the passway, good connection between the old and new sheet piles and the sheeting of the coffer may not have been obtained; the material used for filling the passway coffer when rebuilt may have contained a large proportion of silt, as the silt deposit in the coffer during the winter was very heavy; the seepage probably gradually increased with the increasing head until, during the night, the little stream began to carry out sand from beneath the coffer, and it also probably cut down into the inside banking until the limiting plane of saturation was reached, when the blowout occurred. It is certain that there was a considerable increase of flow during the night from this and probably other seepage streams, and that at 6:30 a. m. there was a bad leak under the passway, and when the blowout took place the whole mass became suddenly fluid and lifted the sheet piles and the coffer.

The Harrisburg Filter Plant made a new high record for bacterial removal during 1912; the average efficiency for the year was 99.97 per cent. The Susquehanna River water contained, on the average, 5300 bacteria per cubic centimeter, while the treated water showed average counts of only 2. The operating expenses, at the plant, which includes coagulating basins, rapid filters and hypochlorite apparatus, were as follows, per million gallons of water treated: Coagulant, $1.35; coal, 36 cents; supplies, 42 cents; repairs, 52 cents; oil and waste, .05 cent; chemist and laboratory, 37 cents; salaries, $2.46; total, $5.53. The fixed charges for interest and sinking fund and state tax on the filtration loan was $6.94 per million gallons. The operation of the filtration plant is in charge of Mr. George G. Kennedy. Mr. James M. Caird has served as chemist and bacteriologist.

Sterilizing Water with Ultra-Violet Rays

Description of New "Pistol" Light and Its Applications in Large and Small Scale Plants

By M. von Recklinghausen, Ph.D., Westinghouse-Cooper Hewitt Company, of Paris

Within the last two years a great deal of work has been done in Europe in the development of ultra-violet ray apparatus for water sterilization. The laboratory work which I have conducted jointly with Messrs. Henri and Helbronner in Paris has been followed by a good many industrial applications of the type of apparatus described in the Engineering Record of Dec. 10, 1910, and July 15, 1911. I think we can safely say that the ultra-violet ray sterilizing system is now an industrial process. There are two distinct parts to the problem—the sterilization of water on a large and on a small scale.

Clear Water Essential

The treatment of water with ultra-violet rays on a large scale generally requires preliminary clarification by means of filters. Filtration at three to ten times the speed of normal biological sand filtration for the same water is sufficient to free the water of matter in suspension. An existing biological filter could multiply its output to a very great extent and produce a better effluent if combined with ultra-violet ray treatment. Once the water is free from suspended matter the speed and the depth of flow past the ultra-violet light have to be fixed according to the specific transparency of the particular water.

The Engineering Record of Dec. 10, 1910, described the type of apparatus which passed the official tests at Marseilles in competition with the different typical purifying processes now known. Such apparatus have now been running at different places in Europe; one plant, at Maromme-les-Rouen, has been in continuous service for two years and a half and gives constantly sterile water. The water is sand-filtered at a speed of about 50 ft. per twenty-four hours and is then passed at the rate of 130,000 gal. per twenty-four hours through the rays of a 1-hp lamp.

For large city plants, however, a 1-hp unit is too small. I was led, therefore, to undertake the construction of larger units with the following objects in view: To locate the lamp as close to the water as possible; to use as much of the light as possible; to produce the ultra-violet rays with a minimum of electric energy; to run the lamps so that they would have an economic life; and finally, to stir up the water during its exposure so as to turn over all microscopic particles which might allow bacteria to hide in their shadows.

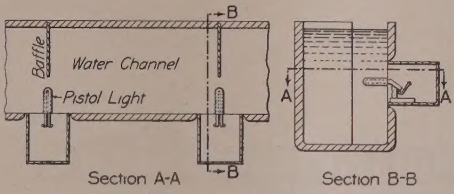
Theoretically we need an extremely short exposure, scarcely more than a tenth of a second at a distance of 1 in. from a lamp, to sterilize water completely. It is safer, however, to lengthen the period of exposure, as is the practice with apparatus described below.

New "Pistol" Lamp

The feature of the latest apparatus is the "pistol" lamp, so called because of its shape. It runs on 500 volts and requires 3 amperes; its peculiar luminous tube is U-shaped, the two branches of the U being very close together. The entire luminous part of the lamp is enveloped by a 2-in. quartz tube, which forms the lamp chamber, separating the lamp from the water. In this new apparatus practically all the light enters the water, while the former apparatus allowed only about 60 per cent of the light to be used.

The efficiency of the new lamp is increased not only by its peculiar shape but also by a very much greater production of ultra-violet light than one would expect from the increased wattage. About ten times as much ultra-violet light is produced in the new lamp as in the old 220-volt lamp, although the wattage is only doubled. Compared with 3.5-ampere lamps the ultra-violet rays from the pistol lamps are fifty times as powerful, although the wattage is only four times as great. Such lamps have been in service in a large European water plant for nearly a year and have given satisfaction.

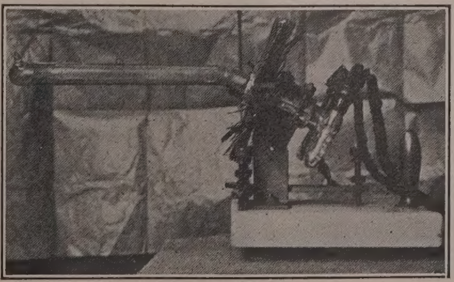
The equipment necessary for such a unit consists of the supporting plate carrying the lamp box, the protecting quartz tube and the lamp itself. The supporting plate forms one of the walls of the sterilizing tank or is inserted into the wall of the sterilizing tank or canal.



Sterilizing Tank with "Pistol" Lamp

For small units of city-water sterilizers—500 to 2000 tons of water per twenty-four hours—only one or two 500-volt or 220-volt pistol lamps are used. For a larger quantity of water there has been developed multiple-unit equipment consisting of a canal of about 50x70-in. cross-section through which the water flows. The canal is made of metal or reinforced concrete and is coated with tiles inside. Into the walls of this canal are inserted ten or twelve of the 500-volt pistol lamps, as shown in the drawing. Baffle plates of glass are inserted across the canal at proper places, so as to keep the water continually stirred up under the influence of the light. An electric water valve, held open by the current of all or some of the lamps, allows water to pass through the canal only when the lamps are burning, so that no unsterilized water may leave the canal. The water in the canal is under the influence of the ultra-violet rays for several minutes, although theoretically an exposure of infinitely less duration would be sufficient.

A canal of this type, with twelve lamps, has been in continuous operation for eight months in a municipal water plant in Europe. The water flows at a rate of about 1,700,000 gal. per twenty-four hours. It is taken from a river and first cleared by a sand-filtering plant. The whole arrangement is extremely simple, and practice has shown that it needs scarcely any attendance. For large water plants the best method of increasing the capacity is to multiply the number of the sterilizing canal units.



Close View of New "Pistol" Lamp

Small-Scale Sterilization

In sterilizing water on a small scale the ultra-violet ray system has to compete with three methods—heating the water, which changes the taste and produces scale; treatment with chemicals or ozone, which is not favorably considered by the public because it changes the taste of the water; and finally, slow filtration through porous porcelain. This latter method gives good results only for a few days. Then bacteria grow through the porcelain and an accidental break of the porcelain tube will produce polluted water.

The apparatus described in the Engineering Record of July 17, 1911, has been applied to a great extent in Europe. One of the interesting applications is for army field work. In this case the equipment, including the necessary water pump, filter and gasoline motor-generator set, is placed on an artillery car. An outfit of this sort may receive any kind of river or spring water and will produce safe drinking water for the troops in the field. One portable equipment was used during the Morocco campaign. In the field hospital where it was used not one case of hospital infection broke out, nor was there a single case of typhoid or dysentery.

A small domestic apparatus has just been developed. It consists of an ordinary U-tube, the necessary cooling of the electrodes being done by inserting the mercury containers partly in the water which is to be sterilized, avoiding, however, contact of the water with the luminous tube itself. This proceeding does away with condensing chambers and enlarged electrodes. It is naturally important that the water level in this apparatus should not change during operation. A constant level is obtained by means of large overflows. This apparatus is designed to produce 20 gal. of sterile water an hour. The lamp requires about 2 amperes on a 110-volt direct-current line.

It is only natural that this new industry has not been developed without difficulties. The most serious one puzzled me for a long while. It was the case of a municipal filtering plant. At irregular intervals it was impossible to obtain good results because algæ were present in the water before they entered the apparatus. It is, of course, easy to avoid such an occurrence. At this plant the bacterial content of the raw water varied from 55 to 655 per cubic centimeter, averaging 197. After sterilization with ultra-violet light the samples showed from 0 to 9 bacteria, averaging 2.5. The apparatus consisted of two 550-volt, 3-ampere pistol lamps. The rate of flow of the water was 440,000 gal. per twenty-four hours.

The Possibilities of Locomotive Fuel Economy have been under investigation on the St. Louis & San Francisco Railroad for the last two years. In a paper presented before the St. Louis Railway Club recently by Mr. R. Collett, superintendent of locomotive fuel service for this railroad company, a chart was shown which indicated the results obtained by comparing the coal consumption in pounds per passenger-car mile and in pounds per thousand-gross-ton mile for corresponding months in 1910, 1911 and 1912. A material reduction was effected in 1911, and this was virtually retained in 1912 in spite of floods, severe weather, suspension of mines and other unfavorable operating conditions occurring during 1912.

Turntable Drawspan for a Temporary City Highway Bridge

To maintain traffic during the reconstruction of the drawspan in a bridge across the Mystic River, between Charlestown and Chelsea, near Boston, there has been installed a temporary plate-girder drawspan about 30 ft. wide and 162 ft. long over all, carrying two electric street-car tracks, and swinging on a pile substructure. This is placed in a temporary pile bridge which is about 1000 ft. in total length, at each end connecting with the old structure. For a distance of about 370 ft. at the center it is parallel to the old structure and 160 ft. from it.

The drawspan has unequal arms, the end bearings being 85 ft. 2½ in. and 72 ft. 8 in. respectively from the center of the draw. The shorter arm is counterbalanced with concrete. At each side of the draw are the main girders, 6 ft. deep and 28 ft. 3 in. apart on centers. The pair of girders at each side of the draw are connected by a pin 12½ ft. from the draw center and the outer ends of the girders are supported when the draw is swinging by eyebars attached to a vertical transverse steel bent about 40 ft. high, placed over the draw center. The draw is center bearing.

The steel floorbeams carry wooden stringers and a wooden plank floor. There are two lines of electric street-car tracks and one sidewalk. The span can be revolved by electricity through 90 deg. in less than one minute, and is provided with hand mechanism by which it can be swung in emergencies.

When opened the span is supported entirely through two transverse bearing girders, web-connected to the main girders, 28 in. each side of the bearing center. The girders are rigidly braced together and are suspended by eight vertical hanger bolts from the saddle casting of the center bearing pivot. The base of the pivot casting has an antimonial lead bearing on a 60 x 60 x ½-in. plate, which is riveted to the top flanges of a set of six 15-in. I-beams 11 ft. long, embedded in concrete. The I-beams rest on grillage of 12 x 14-in. timbers, bolted to caps, which in turn are bolted to the shouldered tops of a cluster of thirty foundation piles, braced by ten spur piles. The caps and grillage extend over and are supported by other piles and spur piles in the fender pier. The I-beams and concrete were placed 2 ft. above mean high water.

Lifting Mechanism

The total weight of the drawspan is about 200 tons. When closed the ends are slightly raised by a hand wheel at each corner, which, through a train of gears and links, drives a roller over an inclined bedplate. A continuation of the same motion places a bearing block and lowers the draw upon it. The bed-plate is made adjustable in various directions

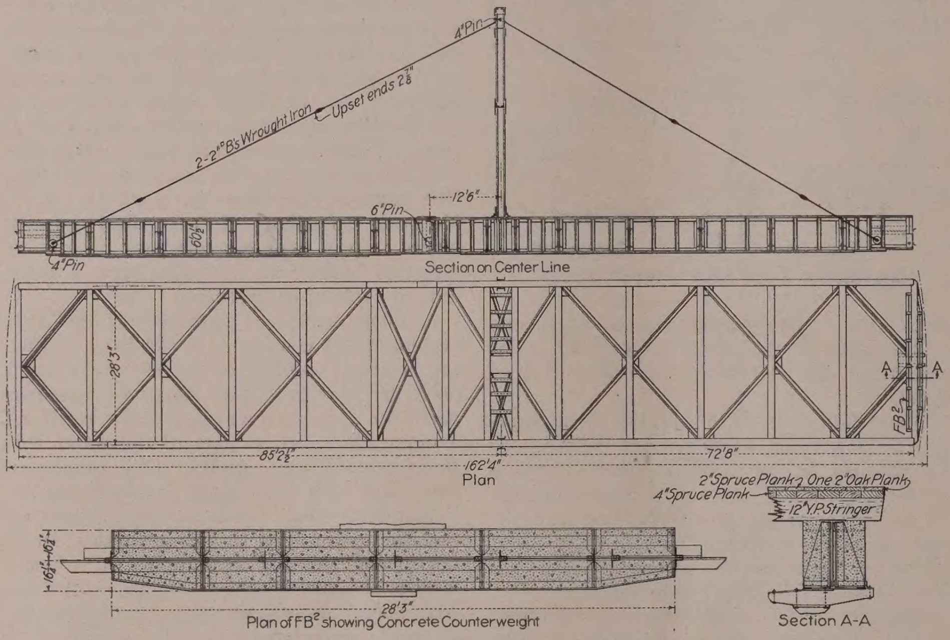
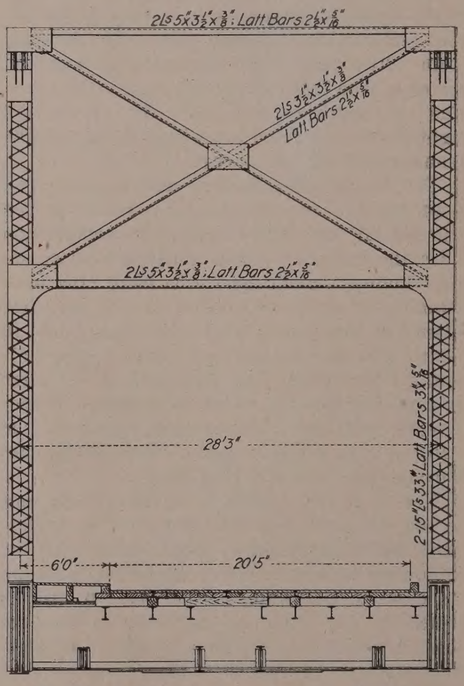
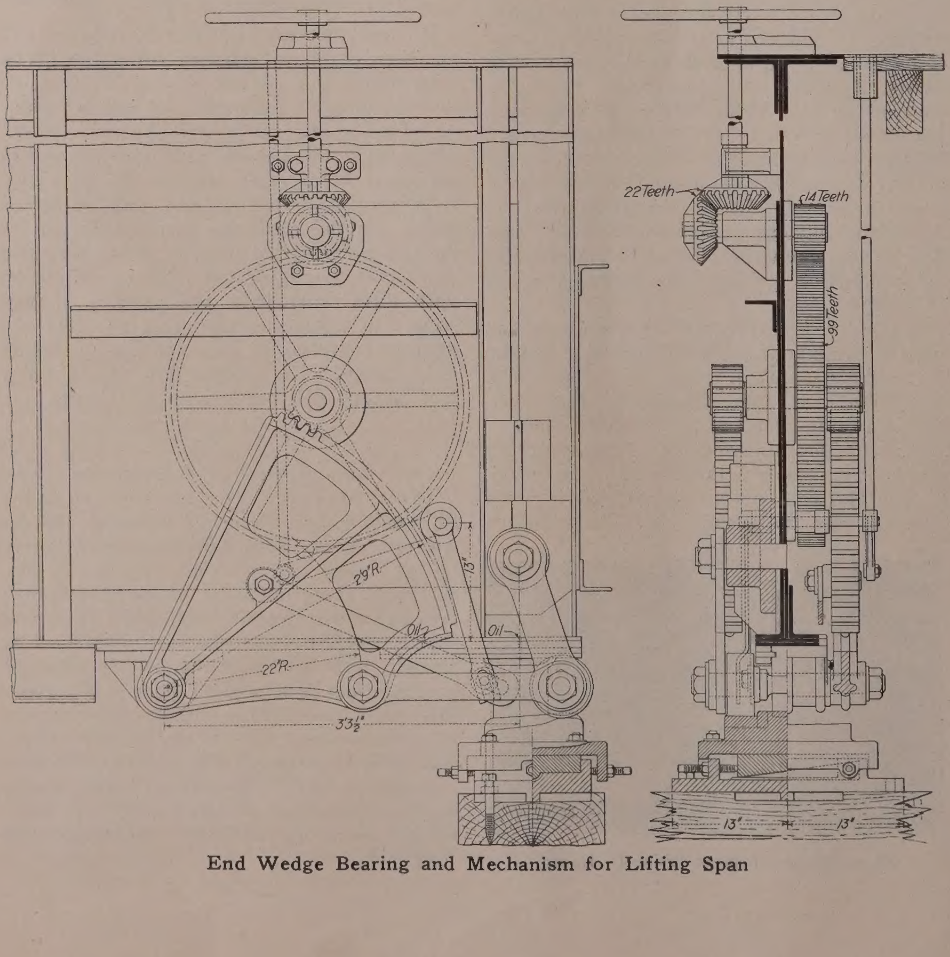


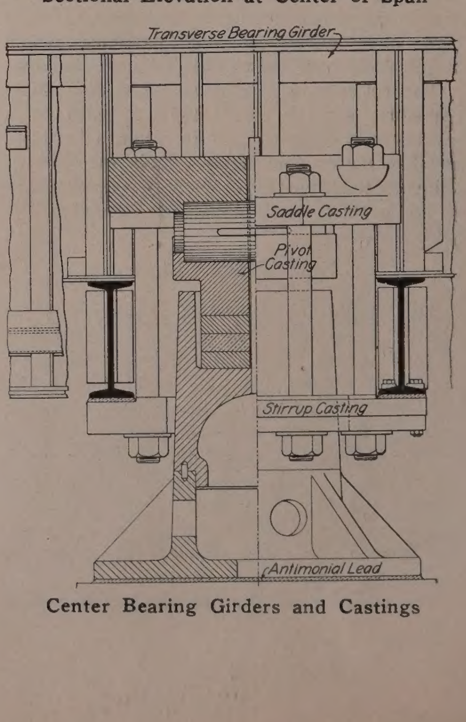
Plate Girder Drawspan and Details of Counterweight


2Ls 5x3½x⅜; Latt Bars 2½x5/16
2Ls 3½x3½x⅜ Latt. Bars 2¼x5/16
2Ls 5x3½x⅜; Latt Bars 2½x5/16
28'3"
2-Ls 5x3½x⅜; Latt Bars 3½x5/16
6'0"
20'5"

Sectional Elevation at Center of Span



End Wedge Bearing and Mechanism for Lifting Span



Center Bearing Girders and Castings

to counteract variations in the pile landing.

The temporary drawspan was furnished for a contract price of $19,194 by Mr. George T. Rendle, contractor for the substructure of the permanent bridge. The work is under the direction of the Public Works Department of the City of Boston, of which Mr. L. K. Rourke is commissioner; Mr. Frederic H. Fay, division engineer, and Mr. S. E. Tinkham, engineer of construction.

New Cement Plant in Vancouver

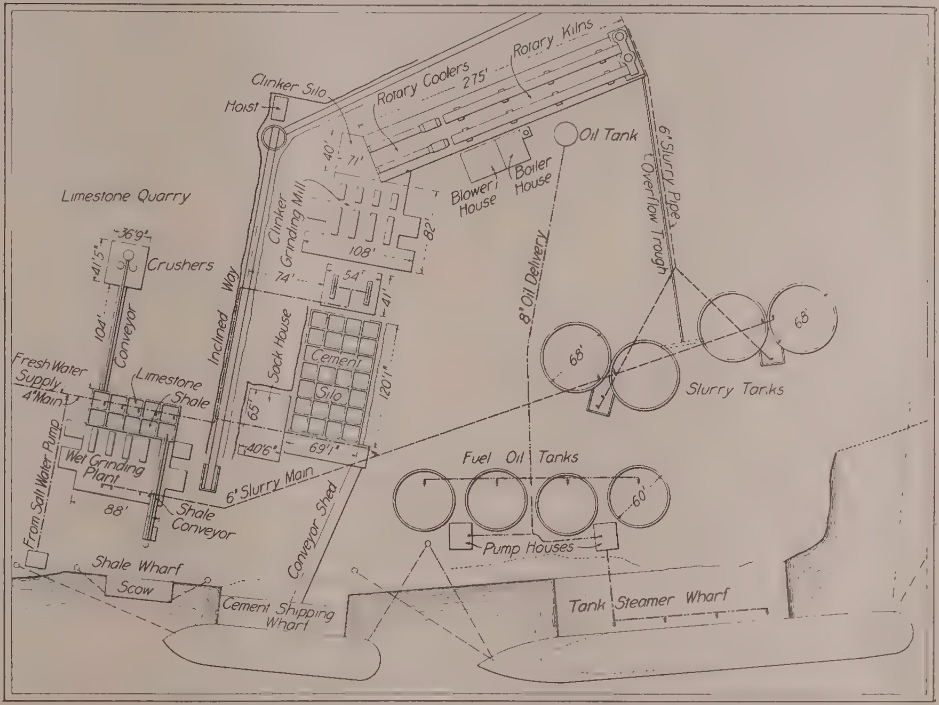
Concrete Sheet Piling and Heavy Construction at Precipitous Site

A cement manufacturing plant having a daily capacity of 2000 bbl., designed, it is said, to employ every process and improvement known to the Associated Portland Cement Manufacturers, Limited, of England, the parent company behind the new development, has just been completed at Bamberton, 14 miles overland from Victoria, B. C. Aside from the introduction of manufacturing methods said to be here used for the first time outside of European plants, the Bamberton works involved special construction features due to the topography of the site. Construction was commenced in January, 1912, and the installation was completed in June, 1913. The crushers, tube mills, conveyors, hoists and, in fact, all fixed driving machines are operated by electricity; practically the only fuel required at the plant is fuel oil for the rotary kilns. In addition to the plant proper it was necessary to build three wharves, erect and equip with pumping plants four 15,000-bbl. steel tanks for fuel oil, and construct quarters for 20 families and barrack accommodations for one hundred and fifty men.

PREPARING FOUNDATIONS

The location of the plant was chosen because of a very large deposit of almost pure limestone, which was high enough above tidewater to permit of being handled by gravity, and was overlaid by only a comparatively thin layer of loose earth and gravel. The first work consisted of clearing the site of a thick growth of timber and excavating for foundations. A total of 160,000 cu. yd. had to be moved before foundations could be placed, and about half of this was rock which had to be blasted.

About 20,000 cu. yd. of loose earth and gravel were sluiced down the hill during the winter season when plenty of water was available at the upper end of the area being worked. In moving material in this manner the sluicing flumes were kept at the toe of the slope being worked, and the loose material was either shoveled into them direct or washed in with the aid of a small fire hose. As the work advanced the flumes were shifted or new branches built, but just below the sluiced area all tributaries emptied into a main flume, 2 x 2 ft. in cross-section, which carried material on a 20-deg. slope to the water's edge. Beyond this point it was unnecessary to give further attention to the material, as the slope of the shore continues below water level to a depth of 700 ft. One main sluicing flume which had its terminus at low-water shore line discharged 17,000 cu. yd. of earth and gravel without perceptibly changing the depth or shore line.



General Plan, Showing Location of Different Parts of Plant

CONCRETE STRUCTURES

A notable feature of the Bamberton plant is the heavy concrete construction used in cement and clinker silos and the buttressed walls of structures built on the steep slope. The heaviest concrete is in the cement silo, which has twenty-eight bins and a total capacity of 60,000 bbl. of cement. This structure, which is 70 x 120 ft. and 70 ft. high, contains 3000 cu. yd. of concrete and 250 tons of reinforcing metal. The bins in this silo being of identical dimensions, the same forms were used repeatedly. One set was made to serve twelve times by using a coating of soap each time before concrete was poured to prevent roughening of the surfaces.

Wherever the ground plan of the buildings was not so irregular as to make it impracticable a unit slab system of construction was used. Slabs 3 x 12 in. x 9 ft. were poured in molds on a casting floor and when set were placed on edge to build up the walls between concrete columns. Each end of these slabs extended 3 in. into adjacent column forms into which concrete was poured after the placing of every four slabs. Each slab had two ⅜-in. horizontal reinforcing rods, mainly to prevent breakage in handling, and all were allowed to set in the molds from three to four days, and were then carefully set aside for three weeks before erecting. Joints between slabs were made tight by a 1 to 1 mixture. This method of construction was adopted to save the labor of material on form work, and is said to have worked out very satisfactorily. In one building, 80 x 40 ft. in plan and 25 ft. high, the cost of the entire building exclusive of roof was 6 cents per cubic foot of inclosed space. The forms for the slabs were used many times, of course, and the average cost of completed slabs, including handling, form work and placing in the building, was about $16 per cubic yard.

Four concrete tanks, with a capacity of

Clinker Silo and Grinding Mill in Course of Construction about 3600 bbl. each, were erected for the mixing and storage of slurry. These tanks are 66 ft. in diameter, and have walls 8 ft. high and 6 in. thick. The reinforcing in the walls is by means of ¾-in. horizontal and ⅞-in. vertical rods. The inside forms were locked together so that when erected the arch action resisted the thrust of the concrete. Wood blocks were used to space outer and inner forms, and the blocks were removed as the work progressed. The outer forms were held in position by means of circular bands. The same set of forms was used for all four tanks.

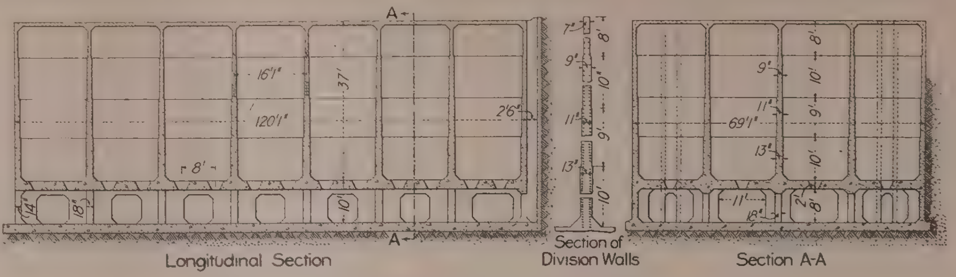
These tanks were the first concrete work undertaken on the job, and the entire concrete mixing gang, with the exception of the foreman, was without experience. The tanks, however, were made watertight without troweling or the use of any compound. When filled with water for construction purposes no trace of leakage developed. The mixture used in the tanks was 1 : 2 : 4, and each batch was carefully spaded in the forms after dumping.

Wharves

The cement shipping wharf is 120 ft. long and stands opposite the silo from which cement will be taken for bagging. It is built of concrete piles, with timber bracing, and has heavy timber struts on the shore side to absorb any shocks caused by vessels bumping into its outer face. The piles are 14 in. square, 45 to 50 ft. in length, and are reinforced with 1¼-in. rods placed in each of the four corners and wrapped with steel wire. These piles were driven 25 ft. into hardpan by a 4500-lb. hammer, and only one pile was broken out of the total one hundred and fifty driven. The cap used in driving was a steel casting with a hemp cushion next to the pile and a hardwood buffer on the top.

These piles were shod with a steel nose 2½ in. square at the tip and tapering to 10 in. square at the point where concrete commenced. This casting was riveted to four ⅞ x 3-in. straps, one of which was embedded in each face of the square pile. The straps were held together in pairs by three ⅞-in. rivets running through the pile at right angles to its axis and connected to the reinforcing.

Shale is delivered to the plant in barges, and in order to bring these in close to the shale bunkers for convenience in unloading it was necessary to provide a retaining wall of some sort to hold back the loose rock on the shore at this point. This was done by driving a line of sheet piling, on the sea side of which a concrete framework was first built to take the thrust from the sheeting. The framework was built up by driving piles 14 in. square 25 ft. into hardpan and connecting these by horizontal whalings. These whalings were placed, after the piles had been driven, by notching out the piles at the required height and placing forms for the whalings so as to connect the opposite notches and give a monolithic framework. From this framework the thrust is taken as tension by means of 2-in. steel rods embedded in concrete beams 6 in. square to a concrete wall built in rock above the tide level.



Structural and Dimensional Details of the Cement Silo

The sheet piles are 8 x 16 in. in section and 30 ft. long. They were driven 10 ft. into hardpan by the use of the same type hammer and cap as already described. Their vertical reinforcing consisted of six 1-in. rods. After this sheeting was in place it was capped with an 8-in. wall 6 ft. high and with its top at the desired level for the wharf deck.

The third wharf is a light structure of wooden piles, used by steamers which dock here while pumping oil to the fuel tanks.

Features of Manufacturing Processes

The crushers are located at one edge of the limestone deposit; in fact, a quantity of limestone had to be removed to make way for the building, and the first rock used practically hung over the crushers. A belt conveyor takes the crushed rock to bins adjoining those where shale is stored after it has been crushed and elevated from the bunkers near the scow landing. Limestone and shale are drawn in desired quantities from the bins, mixed and ground in the wet mill, and pumped from here as slurry through a 6-in. main to the four storage and mixing tanks.

The large storage capacity for slurry is one of the notable features of this plant. It is claimed that all possibility of slight irregularities in slurry composition is removed by thus mixing the product of several days' grinding in tanks which are continually agitated by a patented device. By this means an exact and uniform composition can be maintained, it is said, before the material is submitted to the expensive chemical process of calcination.

Slurry is pumped from the storage and mixing tanks to the upper end of the rotary kilns, connections being such that the supply may be drawn from any tank desired. From these kilns the red-hot clinker passes into the rotary coolers, in which it gives up its heat to the blast of air entering the kilns. After cooling it is stored in the clinker silo, which has sufficient capacity to hold the product of several days' run. The clinker is then ground to such a fineness that 90 per cent will pass a screen having 32,400 holes per square inch.

Bamber Process

During the final process of grinding provision is made for the setting time of the cement by injecting steam into the tube mills together with a much reduced proportion of gypsum. This is known as the Bamber process and is very generally employed in England. Its object is also to hydrate the individual particles and thus remove the expansive properties of any loosely combined lime compounds resulting from slight defects which may have passed the earlier processes uncorrected, and thus produce a cement which may be used immediately after manufacture.

A cooling plant is installed at the Bamberton works through which all cement passes as it leaves the grinding mill, in order that it may enter the storage bins at atmospheric temperature and thus obviate the difficulties engineers often experience from "hot" cement.

The works at Bamberton were designed by the staff of Associated Portland Cement Manufacturers, Limited, under the supervision of Mr. H. K. G. Bamber, Assoc. Inst. C. E. Mr. H. Anderson was resident engineer for the company. The McAlpine Robertson Construction Company, of Vancouver, B. C., which is the Canadian branch of Robert McAlpine & Sons, of London and Glasgow, held the contract for the excavation and the construction of all structures. Practically all of the machinery was manufactured in England and shipped to Bamberton. Power for operating the plant is purchased from the British Columbia Electric Railway Company.

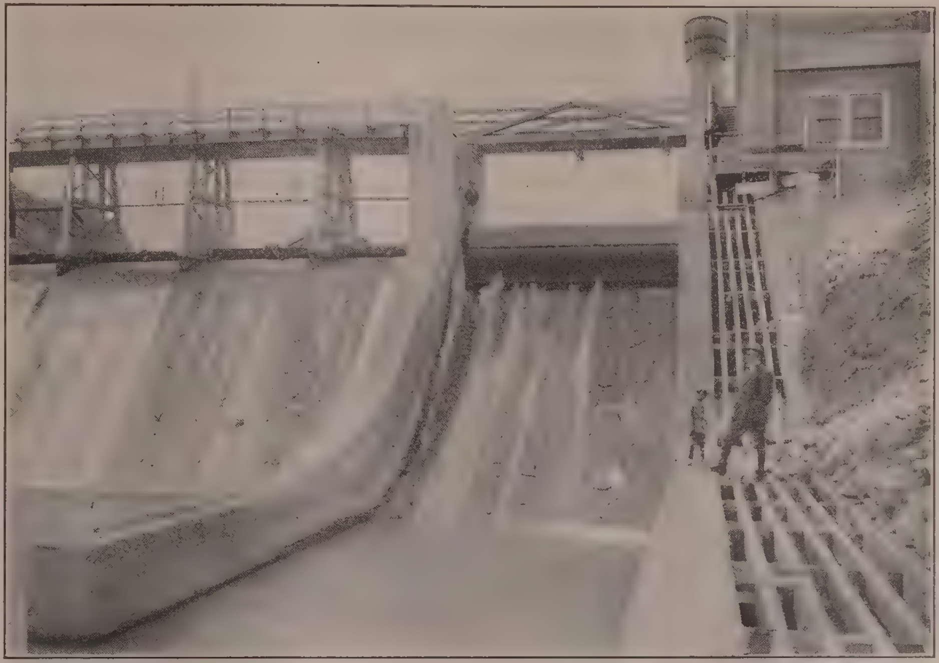
Sack Store Constructed of Cement Slabs

Foundation Reinforcing of Cement Silo

Rolling Dam of the Boise Project

By Charles H. Paul, Construction Engineer, U. S. Reclamation Service, Arrowrock, Idaho

At the south end of the diversion dam for the Boise project of the U. S. Reclamation Service, near Boise, Idaho, there is a logway 30 ft. wide, with its crest 4 ft. lower than the main spillway crest of the dam. It was desirable to install a gate to close this logway, except at times of large floods or when logs must be passed over the dam. The gate most suitable for this purpose is one that closes the whole opening with one span, that permits overflow during floods, and which can be raised high enough to be entirely clear of the water surface, thus permitting logs and débris to pass under it without danger. Considerable time was spent in attempting to design a suitable gate which could be installed at a reasonable cost, but no satisfactory results were obtained. At the same time the type of rolling dam patented by the Maschinen-Fabrik Augsburg-Nurnberg, A. G., of Germany, was investigated and found suitable. It was also found that it could be purchased and installed at a cost considerably less than that of any other gate considered.



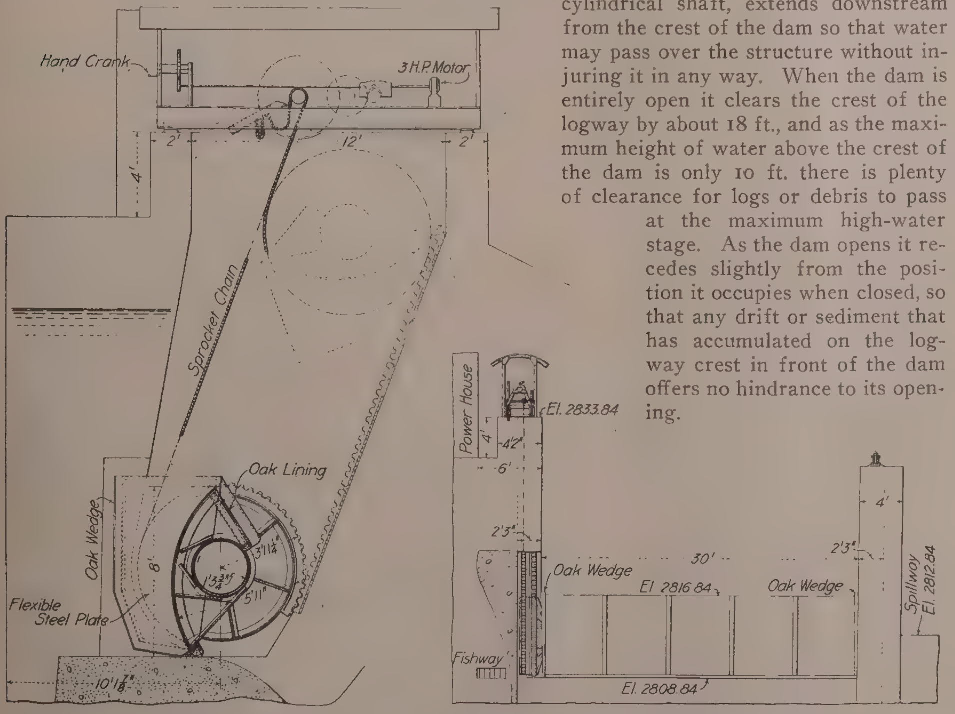
Downstream View of Dam, with Roller in Closed Position

DETAILS OF ROLLING DAM

The rolling dam in its simplest form consists of a large hollow cylinder of a diameter corresponding to the height to which it is desired to raise the water and of a length equal to the width of the opening to be closed. This cylinder is built up of boiler plate and braced sufficiently to stand the strains to which it is subjected. At each end it has a gear engaging a rack laid on inclined abutments and by means of a sprocket chain wrapped around one end of the cylinder and connecting with the operating mechanism the dam is rolled up or down the abutments as desired.

The dam described in this paper differs from the simple form in that the main cylinder is made smaller in diameter, and to it is attached, by suitable bracing, the dam proper, a cross-section of which is the arc of a circle with a 6-ft. radius having a chord length of 8 ft. This arc is not concentric with the main cylinder or shaft, but its center of curvature is located 4 ft. downstream from the center of the shaft and about 9 in. below it in elevation when the dam is closed. The dam is 30 ft. long, corresponding to the width of the logway. The main cylinder or shaft is about 2½ ft. in diameter, and the gears at each end of it are 7½ ft. in diameter. The racks are laid at an angle of 21½ deg. with the vertical.

The inside faces of the concrete piers forming the abutments converge slightly upstream, and at each end of the dam is a flexible plate bound with oak timbers, which spring against the abutments as the dam is being closed, thus making it practically watertight at the ends. An oak sill at the bottom of the dam proper rests on the crest of the logway when the dam is closed, thus making it virtually watertight at the bottom. An inclined timber facing, tangent to the cylindrical shaft, extends downstream from the crest of the dam so that water may pass over the structure without injuring it in any way. When the dam is entirely open it clears the crest of the logway by about 18 ft., and as the maximum height of water above the crest of the dam is only 10 ft. there is plenty of clearance for logs or debris to pass at the maximum high-water stage. As the dam opens it recedes slightly from the position it occupies when closed, so that any drift or sediment that has accumulated on the logway crest in front of the dam offers no hindrance to its opening.



Details of Boise Roller Dam, Showing the General Arrangement

The controlling mechanism is very simple and consists of a sprocket wheel, a shaft, and the necessary gears, and is arranged so that it may be operated by a direct-connected motor or by hand power. The hand-operating mechanism gives a very slow motion, however, and is intended for use only in case of trouble with the motor.

The motor was not furnished with the rest of the equipment, but a 3-hp, 500-r.p.m. motor has been installed and operates satisfactorily. This motor will open the roller in about fifteen to twenty minutes.

PURCHASE OF EQUIPMENT

Correspondence with the German company gave data necessary for preparing a preliminary design and estimate. The company offered either to furnish the equipment complete ready for installation or to furnish all working drawings necessary in order that an advertisement might be issued in this country and the equipment procured from local manufacturers, in which case a royalty charge would be included in the cost of designs and working drawings. Estimates indicated that the cost of the equipment delivered would be about the same in either case, and as it was considered advisable to have the material manufactured by the designers, who are accustomed to building these dams, a contract was made with them covering all the equipment necessary with the exception of the electric motor.

COST OF ROLLING DAM

Preparatory expense (including installation and removal of cofferdam)	$802.18
Transportation of laborers to and from work	327.57
Cutting, chipping and facing old concrete	1,593.95
Placing concrete	1,271.16
Rolling dam and accessories, f. o. b. Boise	2,307.34
Installation	1,105.94
Purchase and installation of motor	122.50
Grand total	$7,530.64

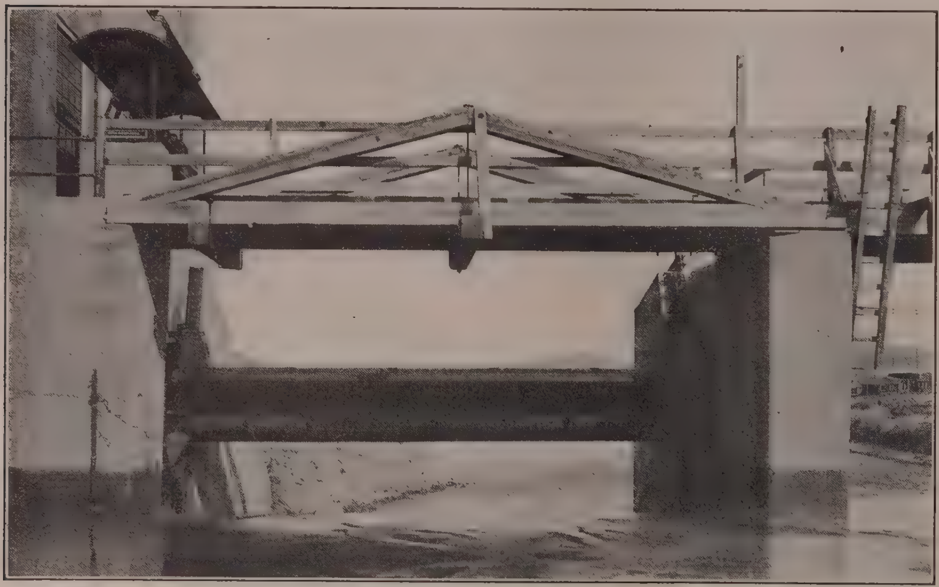
INSTALLATION

In order to adapt the logway as constructed to the installation of this rolling dam it was necessary to cut away part of the south abutment and of the pier separating the logway from the main spillway portion of the diversion dam. This cutting, chipping, and facing of the old concrete constituted a considerable portion of the expense of the preparatory work. About 30 cu. yd. of old concrete had to be cut out in this connection. It was also necessary to increase the height of the abutment and the pier to adapt them to the new requirements and about 75 cu. yd. of concrete were required for this purpose.

On account of local conditions it was necessary to erect a very substantial timber falsework to serve during the installation of the rolling dam.

The assembly of the equipment was accomplished with very little difficulty. The workmanship was exceptionally good. Every rivet and bolt necessary for the complete assembly was furnished and all the parts were labeled in such a way as to aid materially in the work of installation. The drawings and instructions were clear and explicit.

Correspondence with the company leading to the purchase of this equipment was started on Sept. 14, 1911. A contract for the equipment was dated June 12, 1912. Preparatory work was started in August, 1912. The equipment was received Jan. 3, 1913, and the installation was completed during the month of February, 1913.



Upstream View of Boise Dam—Roller Raised About Half Way

The Longest Tunnel in South Africa, the Stockton tunnel, in Natal, is described by Mr. A. Colman, assistant engineer on the South African Railways, in a recent number of the "South African Railway Magazine." The tunnel is 2586 ft. long, through what is considered probably the hardest class of rock in Natal. Headings 9 x 9 ft. were started simultaneously at each end, but one was a top and the other a bottom heading, the former being used because the approach cut at that end was not deep enough at the time to permit a bottom heading. At the south end the top heading was driven as fast as possible, the widenings on each side following close behind, after which most of the bottom section was removed, leaving a wide enough bench along the side for a tramway to haul out the material from the heading. At the north end timber stagings of 12 x 12-in. timbers, with 4-in. flooring, were erected at intervals at the top of the heading after it had been widened. The headings were started about May 1, 1912; they met Feb. 22, 1913, and the excavation is now practically completed. The tunnel will be lined with concrete.

Tests of Reinforced-Concrete Wall and Column Footings

Conclusions Derived from a Series of 197 Tests Made at the Engineering Experiment Station of the University of Illinois

A new contribution to the literature of reinforced-concrete has resulted from a series of tests made at the University of Illinois. The data secured relates to the design of wall and column footings, regarding which there is but little experimental information. The investigations were first undertaken in 1907 in connection with the work of the Engineering Experiment Station and have been pursued more or less continuously ever since. The results are published in a University of Illinois bulletin, No. 67, written by Prof. A. N. Talbot.

The test pieces included 114 wall and 83 column footings. The wall footings were 12 in. wide, generally 5 ft. long and 12 in. thick with a 12 x 12 x 12 in. stem in the middle through which the load was applied. Tests of brick and of unreinforced-concrete footings, and of footings with incased I-beams were included in the investigations.

The column footings were 5 ft. square and generally 12 in. thick, with a 12-in. cube of concrete built at the center through which the load was applied.

Summary of Wall Footing Tests

The tests of wall footings cover a variety of reinforcement. The method used to secure a distributed upward pressure introduced difficulties in testing. It also made it difficult to determine the load which should be taken as the critical load, and the loads which have been so specified may not always be the true critical load. The use of the bed of springs on the whole proved very satisfactory and is probably the best available arrangement for tests of the number and range used. The tests bring out phenomena which might not be apparent from analytical considerations alone or which might not be accepted without physical verification. Variations in concrete add to the complications encountered in analyzing such a series of tests.

1. Wall footings under load follow the general laws of flexure.

2. The values of the modulus of rupture found in the unreinforced concrete footings are not far from the values of modulus of rupture obtained in simple beam tests such as the control beams.

3. The results of these tests and the measurements of deformation of the reinforcement indicate that the critical section may be considered to be at the face of the wall and that the calculated tensile stress in the bars at this section is probably somewhat above the maximum tensile stress developed. Whether the maximum compressive stress may properly be calculated in the same way was not determined. It may be expected that high compressive stresses exist at the intersection of wall and projection. Indications of high compression and of incipient compression failure were found at the intersection of the wall and footing at loads above the critical loads.

Test pieces in which the wall was poured after the footing had taken its set gave results which indicate that a section at the face of wall may properly be used in calculations of moments even when the wall is to be poured separately from the footing.

Bond Strength Important

4. The calculation for bond stress, based upon the total external vertical shear at the section at the face of the wall and calculated by the equation $u = V/mod'$, evidently give stresses higher than the existing stresses. (In this equation, u is the unit bond stress; V is the total vertical shear at any given section; m is the number of longitudinal reinforcing bars; o is the periphery of one bar; and d' is the distance from the center of the reinforcement to the center of gravity of the compressive stresses). This is shown by the fact that the values calculated in this way are higher than those found in pullout tests and beam tests. A study of the analytical conditions existing at this section tends to confirm the statement. However, as bond resistance is so important a strength element in a short cantilever beam, this method of calculation and the use of the working value of bond stress ordinarily assumed in design seems only reasonably conservative and may be recommended for general practice. Attention may properly be called to the importance of making calculation of bond stress in wall footings and other beams in which the length is short relatively to the depth. The advantage of using relatively small bars in such cases is also apparent.

Anchorage of bars by bending upward and back in a long curve or by looping the bar in a horizontal plane was found to add materially to bond resistance.

Diagonal Tension Failure

5. The tests indicate that the vertical shearing stresses developed at the face of the wall, and calculated by the usual method, are higher than the vertical shearing stress which is found to exist in simple beams with concentrated loading when diagonal tension failures are developed. It was found that diagonal tension failures start at a point some distance away from the section at the face of the wall. This observation and certain analytical considerations such as the probable greater proportion of shear taken in the compression area at sections near the face of the wall show that, in calculating the vertical shearing stress which shall be used as a basis for judging

the resistance to diagonal tension, a section some distance from the face of the wall should be used. The tests and the discussion indicate that a section d distant from the face of the wall (d being the distance from center of reinforcing bar to top of footing) may properly be used as the critical section for calculating the vertical shearing stress for this purpose, and that at this section the ordinarily accepted working stress may properly be used for calculating resistance to diagonal tension failure.

6. The bending up of bars at several points along the length of the projection gave added resistance against diagonal tension failure. Vertical stirrups also added to the resistance against diagonal tension failure but were not especially effective. Neither method of web reinforcement would be very convenient in construction. Generally speaking, it will be best to try to design the footing so that the vertical shearing stresses will be within the limit of the working stress permitted in beams without web reinforcement, and thus avoid the use of web reinforcement. In large important footings, when diagonal tension is a critical element, it would seem that some kind of unit frame with well-formed web reinforcement would be preferable to placing stirrups or to bending up bars at the necessary intervals. In stepped and sloping footings attention should be called to the larger diagonal tension and bond stresses developed. The increase in these stresses over those found in footings of uniform depth may be sufficient to decide against the use of stepped and sloping footings.

7. The footings having I-beams embedded in the concrete carried high loads, perhaps corresponding to the yield-point tensile strength of the lower flange of the I-beams and more than double what would be carried by naked I-beams. The weight of the I-beams, of course, was greater than that of the reinforcing bars used in the reinforced concrete wall footings.

Summary of Column Footing Tests

The requirement of uniform load and the presence of double-curved flexure complicate an investigation of column footings. The results may not easily be summarized, but the following statements are intended to cover the principal matters brought out in the tests:

1. A square column footing under load may be expected to take a bowl-shaped form. In slabs subject to bending in two directions, the stress in a fiber can not differ from that in an adjoining fiber at the same level without setting up longitudinal shear; and as there is considerable resistance to variation from equality of stress in adjoining fibers, it may be expected that in stiff thick pieces (as are footings of ordinary design, where the thickness is large in comparison with the length of the projection) the deformations and consequent stresses will be distributed over the width of a cross-section and that considerable stress will be developed even in the fibers at the edge of the footing.

2. For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point half-way out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section. By equating this bending moment and the resisting moment which is available at the given section, the maximum tensile stress in the concrete or in the reinforcing bars may be calculated.

3. As is usually the case when plain concrete is used in flexure, the unreinforced footings show considerable variation in results. The variations were such as not to permit a method of determining the effective width of resisting section to be established or to obtain a formula for resisting moment. Based upon the full section of the footing, the moduli of rupture obtained were considerably less than the moduli of rupture of control beams made with the same concrete.

4. In reinforced concrete column footings, resistance to non-uniformity of stress in adjoining bars will be given by bond and by longitudinal shear in the concrete, and the amount of variation from uniformity of stress in the various bars will depend upon the spacing of the bars as well as upon the relative dimensions of the footing. With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edge of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement spaced uniformly over the footing, the method proposed for determining the maximum tensile stress in the reinforcing bars, is to use in calculation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stress.

Bond Strength

5. The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond stress formula, and to consider the circumference of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section.

An important conclusion of the tests is that bond resistance is one of the most important features of strength of column footings, and probably much more important than has been appreciated by the average designer. The calculations of bond stress in footings of ordinary dimensions where large reinforcing bars are used show that the bond stress may be the governing element of strength. The tests show that in multiple-way reinforcement a special phenomenon affects the problem and that lower bond resistance may be found in footings than in beams. Longitudinal cracks form under and along the reinforcing bar due to the stretch in the reinforcing bars which extend in another direction, and these cracks act to reduce the bond resistance. The development of these cracks along the reinforcing bars must be expected in service under high tensile stresses, and low working bond stresses should be selected. An advantage will be found in placing under the bars a thickness of concrete of 2 in., or better 3 in., for footings of the size ordinarily used in buildings.

Difficulty may be found in providing the necessary bond resistance, and this points to an advantage in the use of bars of small size, even if they must be closely spaced. Generally speaking, bars of ¾-in. size or smaller will be found to serve the purpose of footings of usual dimensions. The use of large bars, because of ease in placing, leads to the construction of footings which are insecure in bond resistance. In the tests the column footings which were reinforced with deformed bars developed high bond resistance. Curving the bar upward and backward at the end increased the bond resistance, but this form is awkward in construction. Reinforcement formed by bending long bars in a series of horizontal loops covering the whole footing gave a footing with high bond resistance.

6. As a means of measuring resistance to diagonal tension failure, the vertical shearing stress calculated by using the vertical sections formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing was used. This calculation gives values of the shearing stress, for the footings which failed by diagonal tension, which agree fairly closely with the values which have been obtained in tests of simple beams. (The formula used in this calculation is $v = V/bjd$, where V is the total vertical shear at this section taken to be equal to the upward pressure on the area of the footing outside of the section considered, b is the total distance around the four sides of the section, and jd is the distance from the center of reinforcing bars to the center of the compressive stresses). This stress is somewhat larger than the average vertical shear over the section which is sometimes used. The working stress now frequently specified for this purpose in the design of beams, 40 lb. per square inch, for 1:2:4 concrete, may be applied to the design of footings.

The punching shear may be calculated for the vertical sections which inclose the pier footing, although it may be expected that shear failure may not be produced exactly on this section. The value now generally accepted for punching shear, 120 lb. per square inch for 1:2:4 concrete, may be used for the working stress in this case.

Compressive Stress

7. No failures of concrete in compression were observed, and none would be expected with the low percentages of reinforcement used. In frequent cases there were signs of distress near the intersection of pier and footing where there is an abrupt change in direction of surfaces and where the combined stresses are very high.

8. In stepped footings, the abrupt change in the value of the arm of the resisting moment at the point where the depth of footing changes may be expected to produce a correspondingly abrupt increase of stress in the reinforcing bars. Where the step is large in comparison with the projection, the bond stress must become abnormally large. It is evident that the distribution of bond stress is quite different from that in a footing of uniform thickness. The sloped footing also gives

a distribution of stress which is different from that in a footing of uniform thickness. However, for footings of uniform thickness the bond stress is a maximum at the section at the face of the pier; in a sloped footing the bond stress at the section at the face of the pier would be less accordingly than in a footing of uniform thickness, and a moderate slope may be found to distribute the bond stress more uniformly throughout the length of the bar. This is not of advantage if the full embedment of the bar is effective in resisting any pull due to bond.

9. The use of short bars placed with their ends staggered increases the tendency to fail by bond and cannot be considered as acceptable practice in footings of ordinary proportions. In footings in which the projection is short in comparison with the depth the objection is very great.

10. Footings having reinforcement placed in the direction of the diagonals as well as parallel to the sides (four-way reinforcement) gave good tests.

Conclusion

The tests of wall footings and column footings leave uncertainty in some parts of the problem and there are gaps in other parts. The recent development of the portable extensometer or strain gage and the skill and experience which have been gained in its use in recent tests have opened opportunities for obtaining information on the stresses developed in such test pieces which were not available when the series of tests was undertaken. It is suggested that some of the remaining unsolved problems may most readily be attacked by measurement of deformations in the steel and concrete, and that further investigation may best be carried on by constructing a form of apparatus which will permit such measurements to be taken under the conditions of uniform loading.

Wave Action has caused a constant wearing away of the shores of the Wachusett reservoir of the Metropolitan Water Board in Massachusetts. The erosion has been especially severe in places where there were steep banks composed of fine material. By this caving, according to the board's report for the year 1912, the area of the reservoir has been increased to the extent of about 14.5 acres, requiring in each year considerable additional stripping of the soil back of the immediate shore, reinforcement of various kinds, and the repair of paving and riprap.

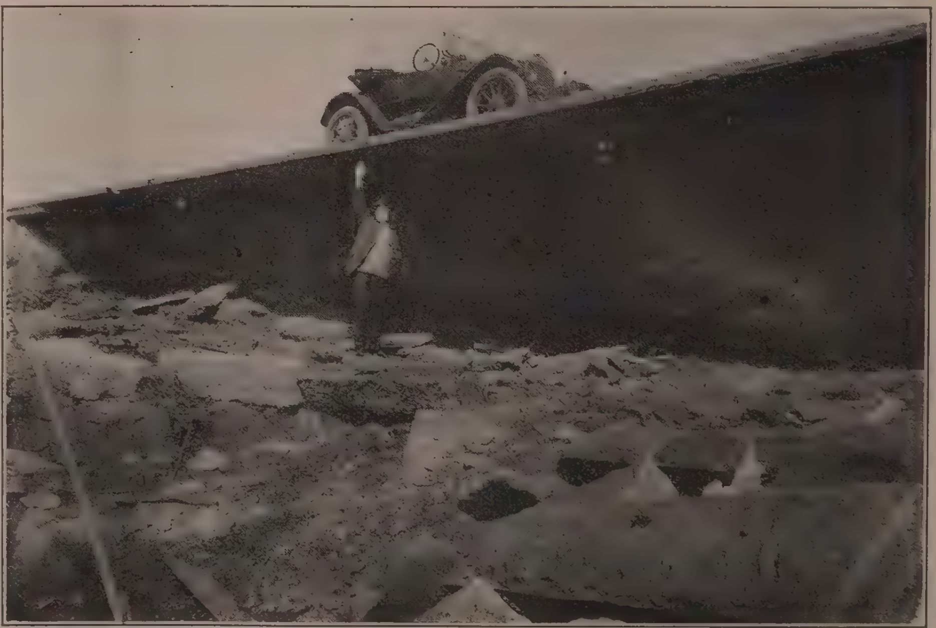
Brick Pavement, after Flood, Acts as Cantilever

A fabled hobo once asked a lady to sew a shirt on his button. The case has a parallel in Cuyahoga County, Ohio, where county engineers will shortly attempt to put a road under their pavement. The necessity for this novel form of construction was occasioned by a flood in Tinker's Creek, south of Cleveland. The force of the water was great enough to wash away a heavy concrete retaining wall which supported the side of an earth embankment carrying a brick pavement, 14 ft. wide, laid upon a concrete base.

Extent of Damage to Retaining Wall by Flood Water

After the retaining wall had gone out the flood waters attacked the embankment and undermined the pavement for a width of 7 ft. and a length of 80 ft., washing out several hundred cubic yards of earth and moving large boulders. The concrete base of the pavement broke away, but the brick surfacing, which had been bonded with a cement-grout filler, remained intact, spanning the gap as shown in one of the illustrations. For several weeks this section of pavement has been acting as a cantilever with a 7-ft. overhang and has been strong enough to support an automobile.

It was first planned to break down the overhanging portion of the pavement and relay it, but after the brick had withstood some hammering from a heavy maul it was decided, reports state, to refill beneath the pavement with earth, tamp concrete laterally under the brick and above the refilled earth, and replace the concrete retaining wall.



Brick Pavement Acting as Cantilever after Washout of Foundations

Electric Well Drills in a Cement Quarry

A cement quarry opened several years ago on the bank of the Hudson River near Glens Falls, N. Y., was worked for approximately three years before there was sufficient space between the quarry face and the river to throw a big tonnage of stone. During that time small blasts were necessary. For this purpose two tripod drills, each operated by two men, were used steadily to drill 1½-in. holes from 12 to 14 ft. deep, furnishing the mill with approximately 650 tons of stone per day. As the work progressed away from the river the height of the ledge increased to 65 ft. and it required five courses of blasts to face the rock from top to bottom. Many different methods were tried, but none effectually dislodged the broken stone from the shelf.

The extra work required to clear away the broken stone caused the quarry cost to run rather high and it was decided to try blast holes of greater diameter and depth as soon as there was space enough between the ledge and the river to shoot off the whole face of the rock. Mr. F. L. Jorgensen, superintendent of the Glens Falls Portland Cement Company, describes the outfit used for this purpose as a Keystone well drill equipped for 6-in. holes and driven by a variable-speed induction motor. Connections to the motor are made with the power line passing near the quarry at three different poles on which the line is strung, the slack of the motor wire being rolled over a drum. As the electric current was already available in the quarry it was estimated that a considerable saving over the cost of operating a steam-driven machine could be made by its use. Also, the services of two men that would have been required to run a steam-driven machine and the extra expense of hauling coal and water were eliminated.

It was thought that only one man would be needed as a drill runner. A steady, reliable laborer who had never seen a well drill before and who had never made more than about $2 per day was selected. An experienced drill runner was borrowed for a few weeks from the Keystone factory. His work while training the inexperienced man showed that a footage of from 30 to 35 ft. per day, including moving the machine from hole to hole, might be expected under ordinary conditions. With this as a basis, the new drill runner was placed on a contract basis at 9 cents per foot of holes drilled, all equipment to be furnished him.

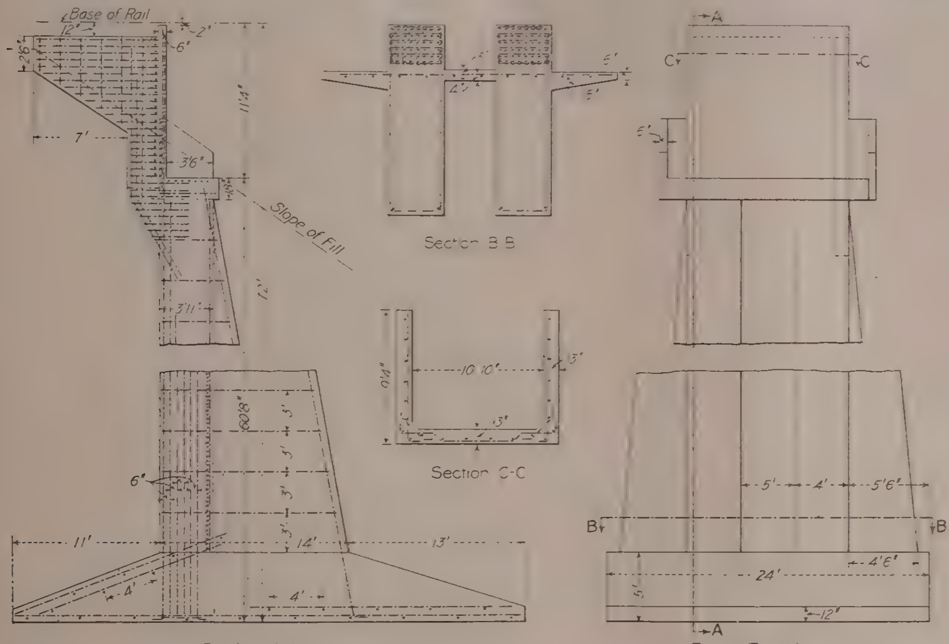
The material drilled is all stone, the overburden being stripped off. The rock consists of about 75 per cent hard limestone, the remainder being fairly soft slate. The total cost for drilling is reported by Mr. Jorgensen to average less than 15 cents per foot, which includes repairs, oil, power, driller, etc. The actual reduction in the quarry pay roll, due to the adoption of this method, is said to be between $12,000 and $15,000 per year. Of this sum approximately $2,000 per year is saved on the cost of drilling, while from $10,000 to $13,000 is saved in bench-cleaning, track-moving, etc. To supply 650 tons of stone per day the drill has to work only half time. With the tripod drills the 1½-in. holes had been drilled 6 ft. back from the face of the quarry, 6 ft. apart and 12 ft. deep. With the new equipment the 6-in. holes are drilled 20 ft. back from the face, 20 ft. apart and the total depth of 65 ft. to the bottom of the quarry. Approximately 500 lb. of 40 per cent Red Cross dynamite are used in each hole and from four to twelve blasts are shot at a time. The rock is reported as being broken up just as well by the new method as by the old one. No increase in the consumption of explosives has occurred either in the initial blasting or in the breaking-up of the big boulders. The loading of the stone is done by hand as piecework. The larger boulders are broken up for the loaders, who then break up and load the smaller pieces into cars, for which work they receive about $2.50 per day at a rate of 10 cents per ton.

The prospect of heavy blasting in the quarry somewhat disturbed the serenity of the residents of the neighborhood. One resident, located about one-half mile distant, was fearful of the result upon the output of his incubator. No shock or shaking took place and the chicken man hatched his eggs as usual.

Features of Louisville & Nashville Work between Nashville and Birmingham

Some of the Characteristic Designs and Construction Methods Being Used

As stated in the article on page 97 of the issue of July 26, 1913, on the general features of the extensive Louisville & Nashville improvements now under way, the north end of the series of main-line changes is at Madison, Tenn., about 9 miles north of Nashville. Leaving the main line at Maplewood, about 3 miles south of Madison, a new cut-off descends to the Cumberland River bridge, and then begins an ascent on the ruling grade which extends beyond where it rejoins the main line. The principal features of this cut-off are the Cumberland River Bridge and the Radnor yard. The reinforced stub abutments used in the bridge are of a type adopted by the company in a number of its bridges. The details of one of them are shown.



Cumberland River Bridge Abutment—Typical Design for Viaducts

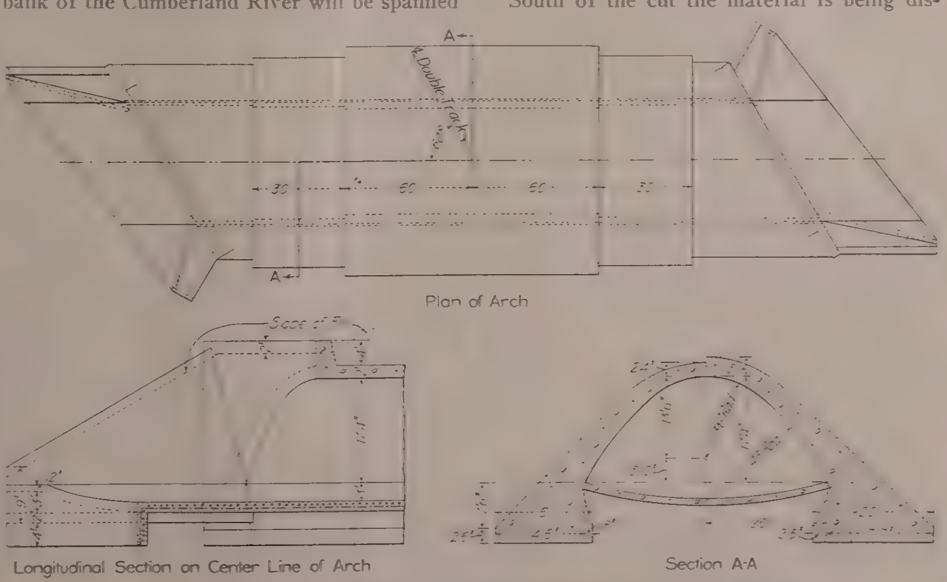
Three railroads are crossed by this cut-off. The Tennessee Central Railroad on the south bank of the Cumberland River will be spanned by the new bridge over the river, the new line being 75 ft. above the Tennessee Central. The single track of the Lebanon branch of the Nashville, Chattanooga & St. Louis Railway will be crossed at grade. At the point of crossing the main line of the Nashville, Chattanooga & St. Louis this road has two tracks in a rock cut about 30 ft. deep surmounted by about 12 ft. of earth. The new road will span the old practically at right angles with a 60-ft. arch bearing directly on the solid rock. The base of rail of the cut-off line will be about 50 ft. above that of the tracks of the other railroad.

BRENTWOOD CUT

From Overtons, where the cut-off rejoins the present line, on the 3 miles to Brentwood, where the longest new location turns off, the curvature is being materially reduced, the old line being crossed several times. The ascent from the Cumberland River is continuous on the ruling grade to Brentwood, and in order to hold to this grade it was necessary to cut the existing summit at that point about 40 ft. deeper. The old cut had a maximum depth of about 25 ft. The new cut will be 68 ft. deep at one point and about 4400 ft. long, and will involve about 300,000 cu. yd. of excavation, being one of the largest on any of the Louisville & Nashville work. It parallels the old location as closely as possible without disturbing the present track, and has a curve at each end, with a short tangent between them.

This cut is being excavated by the Walton-McDowell Co., of Nashville. The material is almost entirely solid rock, and is being handled with two Bucyrus 70 steam shovels, with narrow-gage dinkeys and side-dump cars. Two highway crossings are being maintained over the cut. For one, affording access to the Brentwood station, temporary timber bents are used and shifted back and forth as the steam shovels advance. The other highway was already being carried over the present track, and would not permit much shifting. Two plate girders long enough to span the entire cut, intended for ultimate use in one of the bridges farther south, have therefore been pressed into temporary service.

South of the cut the material is being dis-



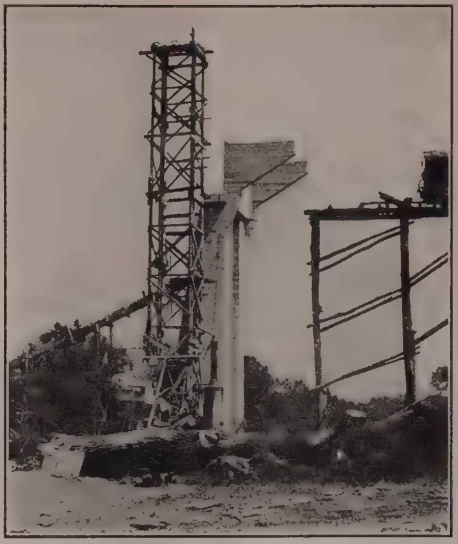
40-Foot Arch at Bridge 19, with Foundations on Loose Rock tributed chiefly in a long fill about 1 mile from the cut. North of the cut, however, the new profile called for no embankment large enough to utilize much of the material. The old and new lines spread here, and between them a trestle has been built on the hillside and a large amount of rock has been wasted off it. A spur track is being run in at the base, and a crusher plant is to be installed to utilize the rock for ballast. Along here, also, the new location fouled a highway, and it became necessary to relocate, grade and macadamize it for 3000 ft.

NEW 100-MILE LOCATION

Half a mile south of Brentwood the old profile comes down to the new; here the new location turns off eastward. From Brentwood to Athens the work is fairly heavy, but not excessively so except on a few short reaches, notably the 0.9 per cent pusher grade south from the Elk River. The heaviest part of this work is that shown in the first article on the profile from Lewisburg to the summit of the pusher grade. The lightest part is that from this summit to Athens.

There are two tunnels on this location. The longer one, 11 miles south of Lewisburg, is 1400 ft. long and has an approach cut at the south end of 195,000 cu. yd. Some of the material goes into fills 1½ miles away. No special difficulties have been encountered in these tunnels.

There are ten open bridges on this new location, the principal one being that over the Duck River, with three spans and a total length of 380 ft. In addition there are a large number of plain and reinforced concrete arches of various sizes. One of the largest of these is the 80-ft. mass arch over Richland Creek. This is a three-centered arch, the middle and end inside radii being respectively 37 and 56 ft. The ring is 4½ ft. thick at the top and 10 ft. at the springing line; there are no footings, the thrust being directly on the solid rock. The arch is built for two tracks, and is normal to them; it is 60 ft. long and has thin reinforced and buttressed end and wing walls. The creek channel was changed to bring the crossing at right angles, and the wing walls are, therefore, of unequal length.

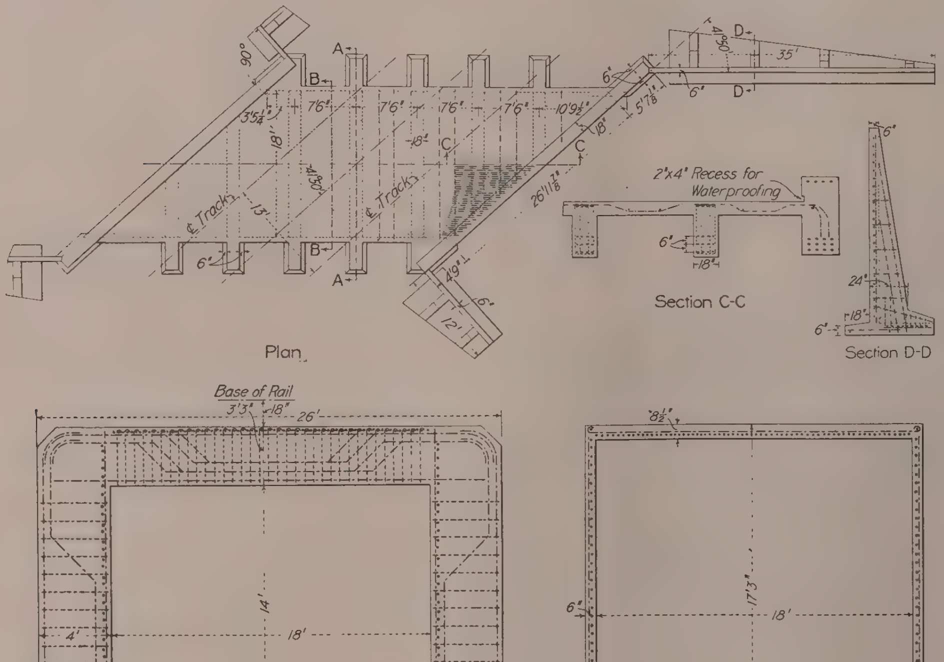


Cumberland River Bridge Abutment

SAND MOUNTAIN PUSHER GRADE

Ignoring the 40-ft. hard-rock cut of 142,000 cu. yd. at Hartselle, on the contract of the C. G. Kershaw Contracting Company, of Hartselle, Ala., the material from which is for the most part being hauled 1 mile north to a fill of 250,000 cu. yd., the heaviest work south of Decatur begins at the pusher grade up Sand Mountain from Wilhite to Holmes Gap. This section was originally built with numerous short curves, several of them being reverse curves. It followed up the ravines of Big Flint and Billy Flint Creeks, crossing the latter twelve times, for the most part on open bridges.

A mile south of Wilhite the old line cut sharply to the left through a thin steep-sided nose of rock, crossed Big Flint Creek on Bridge 19, part of which is shown in one of the illustrations, and reversed sharply to the right, avoiding another rock bluff. The improved alignment cuts deeper into the first nose, crosses the existing center line on the bridge, and cuts heavily into the other hill. The new south cut is 115 ft. deep on the new center line and 145 ft. deep at the upper side. In order to provide more working room the outer rim of the old cut was removed and the existing track moved outward about 15 ft. A Star well digger was let down from the top to drill the holes for blasting, the material being entirely solid rock. The material is being taken out with a Marion 60 steam shovel and 4-yd. Western side-dump cars hauled by Vulcan dinkeys. It is being hauled out on a temporary trestle alongside the bridge and dumped off to make the solid fill over the concrete arch in course of construction.



Flat Reinforced-Concrete Undercrossing South of Cullman

The cut south of Bridge 19, the material from which is to be used to fill over the arch replacing Bridge 20, has not yet been started. Farther up the grade, however, at a point where the new line swings 200 or 300 ft. west of the old one into a heavy fill, another Marion 60 shovel is at work, the material being hauled by standard-gage Vulcan dinkeys in 6-yd. Western dump-cars. A Cyclone drill is at work at the top of this cut. The grading from Wilhite to and beyond this point is being done by M. Jones & Company, of Vinemont, Ala., under the general contract of Hough & Spradlin, of Knoxville, Tenn.

The sub-contractor for the masonry on this section is the J. S. Lusk Construction Company, of Knoxville, Tenn. The new Bridge 19, plans and a photograph of which are shown, is a 40-ft. three-centered arch 280 ft. long. Most of the arches on the work have no footings, bearing directly on the solid rock. At this point, however, only loose rock was found, and the arch ring is therefore being carried down as shown.

In building the arch the materials are dumped from the main track into bins at the end of the bridge and carried down in a chute to the mixing plant shown part way up the bank. As shown in the illustration, the concrete is carried from the mixer in another chute to buggies on a platform level with the top of the arch. Water is pumped up from the pool formed above the cofferdams, and the flow of the stream is carried behind the arch.

The new Bridge 20, about 3500 ft. south of Bridge 19, is also shown in detail. This 20-ft. arch is 310 ft. long. It may be noted that the shape of the opening is much different from that of Bridge 19, and that the arch bears directly on solid rock. The method of construction is in every way the same, except that the water is being temporarily carried through the arch by means of a wooden flume It may be noted that the arch barrels decrease in thickness at the ends.

HOLMES GAP SUMMIT

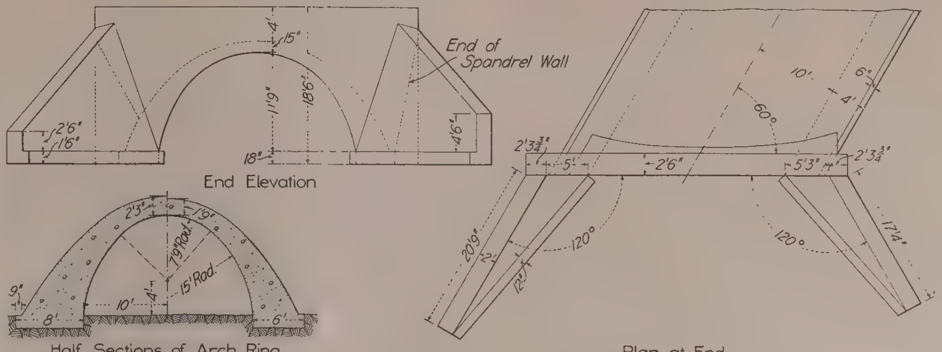
Near the head of the ravine, on a short section being constructed by the Watson Contracting Company, of Birmingham, the line as relocated is directly in the bed of the creek and 3 or 4 ft. below the old track level. To accommodate the creek it was necessary to cut a new channel several feet deep and about 10 ft. wide through the solid rock for several hundred feet. Before the new grade can be put into service the old track and embankment will have to be cut down, and this will necessitate probably further deepening of this channel to protect the new subgrade.

At Holmes Gap, the summit of the pusher grade, and incidentally the highest point on the line between Cincinnati and New Orleans, the new location cuts about 1000 ft. west of the old to avoid a long curve. This necessitated a cut 70 ft. deep and 1900 ft. long. This is being taken out by a Marion 60 shovel at each end in alternate lifts of about 10 ft. Four-yard narrow-gage Kilbourne & Jacobs side-dump cars, hauled by dinkeys of various makes, carry the material. That at the north end is being largely wasted. A short distance from the south end the new line crosses the old on a fill and crosses back about 1000 ft. farther. The contractors, E. K. Langhorne & Company, of Vinemont, Ala., were allowed to put in a frog to cross the main track and make the new fill. This fill has been completed and the spoil from the summit cut is now being carried through a smaller cut to an embankment 2000 ft. long and up to 40 ft. high. The crossing frog has been removed and the narrow-gage track kept to the west of the main track.

Grade Separation Through Cullman

At Holmes Gap there begins a descent on the ruling grade to the Mulberry Fork of the Warrior River near Garden City, a distance of 20 miles, with only two breaks, each less than 2000 ft. long. The old line passed through the village of Cullman at ground level, between two of the main streets, and crossed seven streets at grade. The alignment is not changed here, but the new grade is from 5 ft. lower at the north end of the village to 20 ft. lower in the heart of the town. The most northerly street is cut down and retained as a grade crossing. The next one is closed so as not to disturb the side track to an industrial plant on the high level. The other five are being carried over on three-span reinforced-concrete bridges of standard design, 22 ft. clearance over top of rail being obtained with only slight changes in the existing street grades.

Some difficulty was encountered in convincing the village authorities of the desirability of this change, but they were shown that the village was getting its grade crossings eliminated without expense to itself. Furthermore, the railroad company is replacing the old frame combination station with an ornamental brick station and a separate brick freight house 400 x 40 ft.



End Elevation

Half Sections of Arch Ring

Plan at End

20-Foot Arch at Bridge 20, Bearing Directly on Solid Rock

As stated in the first article, where the highways are carried under the tracks circular arches are being used wherever there is sufficient head room. Two miles south of Cullman is a crossing where there is not room for such an arch, and hence the flat underpass shown in detail is being built.

In this vicinity some trouble has been experienced with sliding material. At Hiram Creek, where a 24-ft. arch was built under a 65-ft. fill, one of the thin reinforced wing walls was broken off by a slide, not from the new embankment, but down the steep slope of the ravine. At another point the new location overlapped the old one for a short distance. The new grade was to be 15 ft. higher than the old; but in order not to introduce into the line under operation a temporary grade that would require additional power it was decided to cut down to the old profile on the new alignment, swing the existing tracks to the new location where they overlapped, and lift the track back the 15 ft. after the entire new line had been put into service. After the track was shifted to the new cut a slide occurred, and it was necessary to remove the material with a steam shovel.

22-Mile Relocation in Northern Alabama

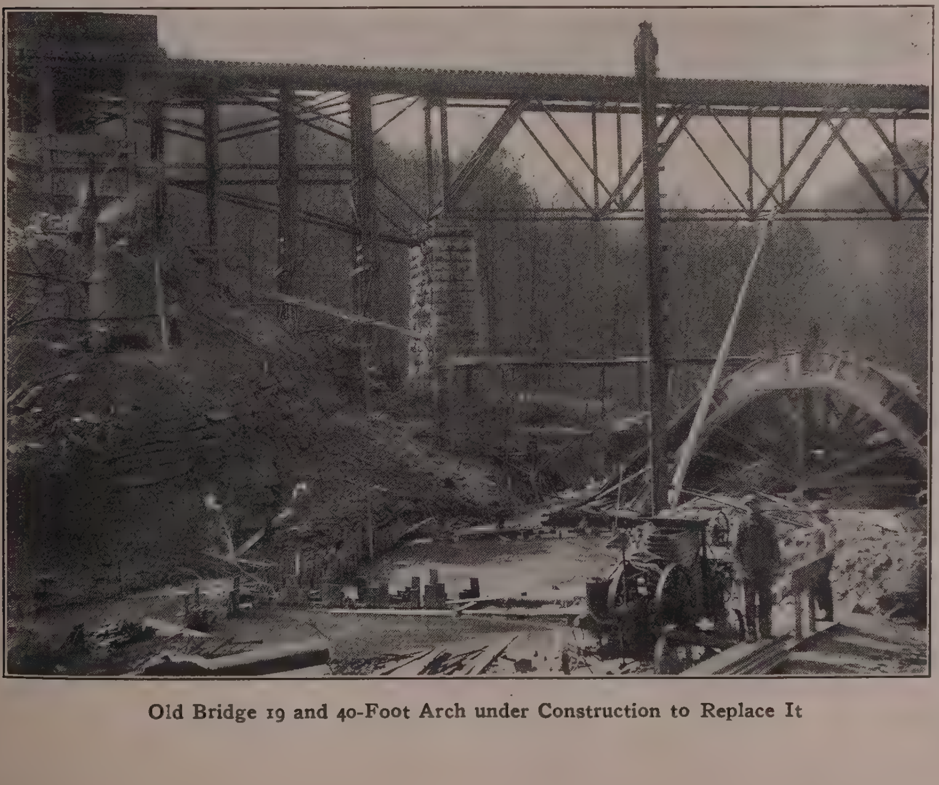
From the Mulberry Fork of the Warrior River, which is crossed on a new location by a five-span bridge 600 ft. long, the profile is broken and the grading is heavy. The contractors are Adams & Sullivan, of Louisville; Thrasher & Gunter, of Knoxville, Tenn.; the Callahan Construction Company, of Knoxville, Tenn., and the Asheville Construction Company, of Birmingham. On the Thrasher & Gunter contract there are two tunnels, one 1000 ft. long and the other 2100 ft. long. There are numerous fills, running as high as 75 ft., which are in general being made from trestles with 8-yd. eight-wheeled side-dump cars of the Oliver and the Western makes. The trestles are made from the tall straight pine trees which abound in the neighborhood, four unstripped trunks being used for each bent, which are about 15 ft. apart.

The Locust Fork of the Warrior River will be crossed with a bridge having a 200-ft. span over the waterway and viaduct approach spans at each end. The concrete work, which is completed, was built by Kreis & Son, of Knoxville, Tenn. The bridge is about 1 mile up the river from the end of an old coal spur from the present line, and this was extended to bring up the materials. The mixing plant was along the track at the bottom of the valley. The concrete for the abutments, piers and pedestals was placed from a 1-yd. bucket traveling on a cableway at about the elevation of the track, which will be 170 ft. above low water. The abutments are of the same type as shown for the Cumberland River Bridge.

This bridge, with that over the Mulberry Fork of the same river, constitute the principal bridges on the Alabama work. The steel in the bridges throughout the work in Tennessee and Alabama is being furnished by the Mount Vernon Bridge Company, the Louisville Bridge & Iron Company, and the American Bridge Company, and erected by the Foster-Creighton-Gould Company, of Nashville, Tenn.

Personnel

As stated in the first article, the work is being done under the general supervision of Mr. John Howe Peyton, chief engineer of construction, assisted by Mr. W. E. Smith, superintendent of construction, and Mr. H. C. Williams, assistant chief engineer of construction, the plans being prepared in the office of Mr. W. H. Courtenay, chief engineer. Messrs. W. S. Morton, division engineer, and George W. Feagin, division engineer, are in charge of all of the work from Madison to Athens, with the exception of the Radnor yard, which is being built under the direction of Mr. E. L. Trowbridge, assistant engineer of construction. Mr. T. Q. Harrison, engineer of construction, assisted by Mr. Leigh Talliaferro, has charge of the work from Decatur to Birmingham.



Old Bridge 19 and 40-Foot Arch under Construction to Replace It

To Irrigate the Valley of the Guadalquiver, in Spain, a company has built a dam at a cost of $180,000 and is now planning the construction of a canal. About 24,700 acres will be irrigated. The total length of the canal system, which will cost about $720,000, is estimated at 25 miles.

Concrete Water Tanks for Industrial Plant

Two reinforced-concrete water tanks, one for supply and the other for storage purposes, the former elevated directly above the latter, have been built for the Burson Knitting Company, at Rockford, Ill. It was desired to build a reservoir of sufficient height so that water pumped to it from artesian wells would flow by gravity through a softening plant to the storage tank and from there to the washing machines in the knitting mill. A conical bottom was designed for the lower tank to act as a settling basin. A 6-in. pipe, with its inlet at the apex of the cone, drains to the sewer. The supply for the washers is taken from that portion of the tank above the cone. The water is first pumped from the wells to a pump pit constructed below the floor level of the mill and inside the footings of the tank columns. Then by an electrically controlled pump the water is forced to the upper tank, keeping approximately a constant head.

Design

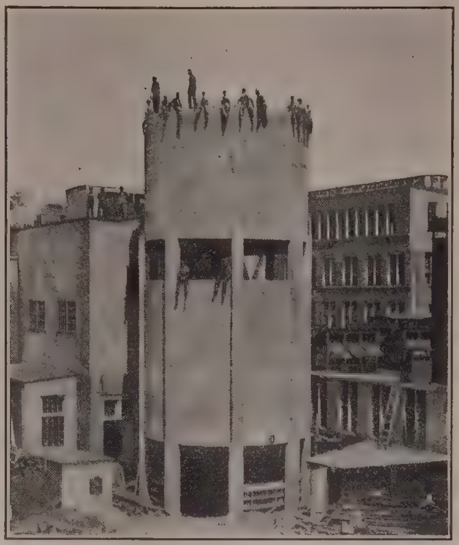
The upper tank, which has a flat bottom, is 18 ft. in height and 24 ft. in diameter and has a capacity of 61,000 gal. The wall thickness is 8 in., reinforced by vertical bars placed in the center of the section and horizontal bars, bent to the radius of the tank, laid between the vertical bars and the outside surface. Four beams, each of 14 x 21 in. in cross-section, are arranged in pairs at right angles to one another on the third points, and supported at the ends by eight 18-in. square columns. The tank floor and beams were laid in one operation, the 7-in. floorslab making a total depth of beam of 28 in. The floorslab is reinforced in each direction by ½-in. bars spaced from 6 to 8 in. on centers. The beams have the usual moment reinforcement and ⅜-in. vertical stirrups spaced uniformly at 6 in. over the entire length of beam.

The lower tank is similarly reinforced, except for the conical bottom, which is strengthened by ½-in. bars placed radially from the apex and circumferential bars of ½ to ¾ in. diameter spaced from 4 to 6 in. The storage tank is 22 ft. high over all and has a capacity of 83,000 gal. The portion of the columns along the vertical walls of the tank was designed as a part of the walls, the horizontal reinforcement passing between the vertical bars in the columns. The columns are widened to 24 in. below the base of the cone and rest upon a footing 8 ft. 6 in. in width circling the entire tank at about 5 ft. below ground line. The annular footing is designed for a load of 1½ tons per square foot equally distributed about each column and is reinforced with circumferential and radial bars embedded near its lower surface. The total height from ground line to top of upper tank is approximately 57 ft.

Erection

The work on the tanks proper was carried on continuously by working two shifts. The forms were moved up by means of jackscrews as the work progressed. The outer wall forms were 6 ft. high and in segments moving between the column forms. The inner forms were of one piece, 4 ft. high, and carried the inner staging. This made it possible to keep the staging at a convenient height from which to work at all times.

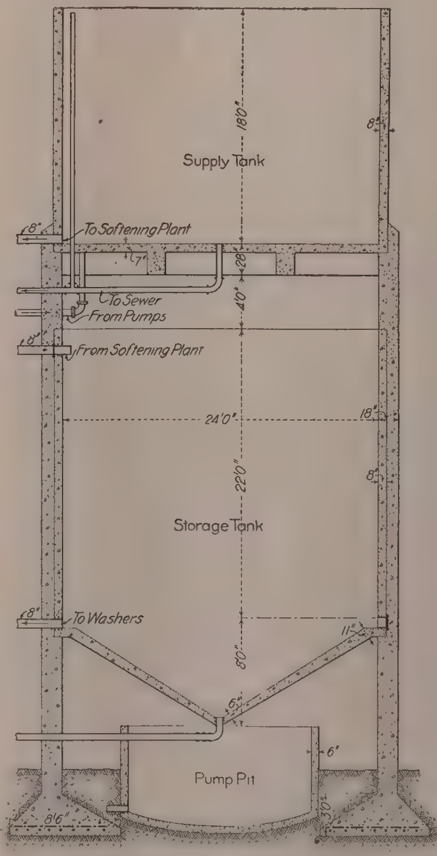
The concrete was hand-mixed in proportions of 1:2:4, the aggregate being gravel which would pass through a 1-in. screen and over a ¼-in. mesh. The distributing was done by means of a swinging pipe chute from a hopper on the outer staging fed by a drop-bottom bucket holding 6 cu. ft. A small portable derrick and an electrically driven friction hoist were used to handle the concrete bucket and all falsework and reinforcing.



Tanks after Removal of Forms

The pipe connections with the tanks were made by threading nipples of length corresponding to the thickness of the wall in which they were to be placed and screwing three disks on them, one on each end and one in the middle. The end disks were tapped for machine bolts, so that flanges of the pipes might be bolted to them, and the middle disk insured a watertight joint about the nipple.

The general layout was planned by Mr. A. E. Burke, engineer for the Burson Knitting Company, and the work was designed and constructed by Mr. Charles C. Stowell, consulting engineer, of Rockford, Ill.



Vertical Section of Superimposed Tanks

Use of the Strain Gage in Testing Materials

Within the last three years rapid progress has been made in the use of the strain gage in testing materials, due largely to its convenience. Its advantages over older forms of extensometers become especially apparent in field tests, and it has been quite extensively applied in measuring stresses in both concrete and steel structures. A paper discussing its use from various angles was read by Messrs. W. A. Slater and H. F. Moore before the meeting of the American Society for Testing Materials last month.

The term "strain gage," as used by Messrs. Slater and Moore, may be considered to mean any kind of an extensometer which during the course of a single test may be used to measure changes of length in various portions of the structure or specimen being tested.

The limitations of the fixed extensometer usually occur in the case where it is desired to find out not the properties of the material in the test specimen, but the actual distribution of the deformations.

Limitations of Fixed Extensometers

The foregoing limitations are due to various causes, several of which are here discussed:

1. In case measurements are required in a large number of places, the cost of fixed extensometers would be prohibitive. Moreover, the difficulty, if not the impossibility, of attaching so many extensometers in such a way as not to interfere with one another would prohibit their use.

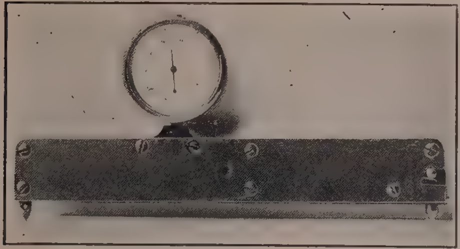
2. In tests of long duration, such as tests of large areas of a floor which requires a considerable length of time for loading, it would be very difficult to prevent disturbance of instruments by persons not appreciating the requirements of great care, and it would be difficult to make temperature corrections. Slight drafts of air also frequently affect seriously the accuracy of readings obtained on fixed extensometers, depending, of course, on the form of extensometer used.

3. Frequently it is difficult or impossible to attach an instrument to the specimen being tested. When attached to the concrete the formation of cracks is often misleading.

4. Most forms of attached extensometers measure change of length not in the extreme fiber of the member but along a line at some distance from the surface, and in order to determine the extreme fiber deformation it is necessary to take measurements on opposite sides of the member. For this method of measurement the time required in preparation for an extensive test would be very great, and the reducing of the data would be complicated—the correctness of measurement at any point would be dependent upon the correctness of the measurement on the opposite side, upon the presence of cracks in the material and upon the correctness of the assumption of "conservation of plane sections." Furthermore, it is sometimes impossible to place fixed extensometers on both sides at a place where it is very necessary to determine stresses. Instances of this sort occur at the interior support of a reinforced-concrete flat-slab floor.

Types of Strain Gage

Two types of strain gage of sufficient precision for the measurement of elastic deformations are in use in this country at the present time, one devised by Mr. James E. Howard, of the Bureau of Standards, and the other by Prof. H. C. Berry, of the University of Pennsylvania.



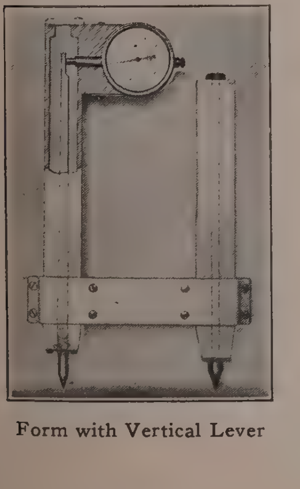
Berry Type with 8-In. Gage Length

The Howard strain gage was used at the Watertown Arsenal as long ago as 1888. It is, in brief, a pair of trammels mounted on a telescoping frame, to which is attached a screw micrometer for the purpose of determining accurately the amount of movement of the points.

The Berry type of strain gage consists of a frame to which are attached a micrometer and two pointed legs; one leg is fixed in position and the other pivoted in such way that it will adjust itself to slight changes in distance between gage holes. Attached to, or continuous with, the pivoted leg is a lever arm which multiplies any movement of the point of the leg. The amount of movement at the end of the lever arm is measured by means of the micrometer attached to the frame. This instrument was described by Professor Berry in the Engineering Record of June 11, 1910. In this instrument the movement of the end of the multiplying lever was measured by means of a screw micrometer operated manually, the instant of contact being determined electrically.

In an instrument of the Berry type designed at the University of Illinois an Ames dial micrometer was substituted for the screw micrometer. This dial is self-indicating and its use decreased the time necessary for taking measurements and greatly facilitated the operation of the instrument, enabling the use of both hands in the seating of the points. In the latest type of instrument developed at the University of Illinois aluminum is used to secure lightness, and a means is provided of varying the gage length at will from 6 to 11 inches.

In one of the figures is shown a newer form of the Berry type of extensometer designed by Messrs. Slater and Moore. It has a very rigid stationary leg and a means of increasing the length of the leg without reducing the multiplication ratio, that is, the ratio of the distance between the pivot of the movable leg and the point of bearing on the plunger of the Ames gage to the distance between the pivot and the point of bearing on the edge of the gage hole. Another feature of the instrument is the use of a multiplying ratio larger than is known to have been successfully used on any other strain gage. It was designed also with a view of obtaining great rigidity in the frame as regards the effect of longitudinal thrust. For this purpose the legs were made rectangular in cross-section instead of circular and to this same object was sacrificed the adjustability of gage length, except for about 1/16 in., which may be obtained by loosening the nut at the top of the right-hand leg and moving the leg in or out a small amount to allow for small variations in the spacing of the gage holes. The necessity for great rigidity in the resistance of longitudinal thrust has been found in all tests where it was necessary for the observer to assume an awkward position in the measurement of deformation. This was particularly true when in a test of the Larkin Building in Chicago in August, 1912, measurements were taken of deformation in steel which had a depth of embedment in concrete of as much as 4 in. Making the multiplying lever vertical, as shown, instead of horizontal, enables the length of leg to be increased without reducing the multiplication ratio.

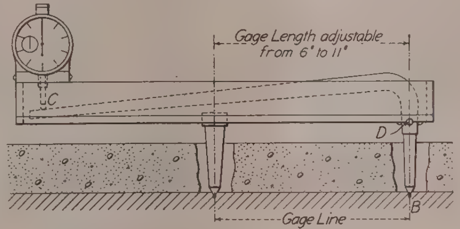


Form with Vertical Lever

Other figures show Professor Berry's latest form of strain gage. It is fitted with a self-indicating gage, has invar steel sides (so that the change of instrument readings due to warming up of the instrument with handling is minimized), and has specially designed pivot bearings. The gage length is not adjustable.

The following method of taking a strain-gage reading has become standard:

Place the point of the movable leg of the strain gage in one gage hole of a gage line, taking hold of the instrument as near the legs as possible; avoiding jerky motion or jar, slide the point of the fixed leg into the other gage hole and read the dial, keeping the points of the legs of the instrument firmly pressed into the gage holes; repeat this process until there have been obtained a series of five consecutive readings which do not vary by more than 0.0003 in.; take the average of this series as the gage reading for an observation. When about to read the gage, it is important to keep the eye turned away from the dial until the instrument is firmly settled in position, else prejudice in favor of the first reading, accompanied by involuntary muscular action, will tend to produce a repetition of a previous reading.



Illinois Form with Horizontal Lever

PREPARATION OF GAGE HOLES

The proper preparation of gage holes and gage lines is a very important factor in securing accurate measurements with the strain gage. The gage holes must be drilled deep enough so that the points of the strain gage cannot touch the bottom of the hole, and the edge of the drilled hole should be smoothed so that all burr is removed, allowing the strain-gage leg to bear against the edge formed by the smoothing action of the finishing tool. It is of prime importance that the gage holes in round bars be drilled as nearly radially as possible, otherwise the readings of the strain gage may be very seriously affected by slight variations in the manner of placing the instrument in position for a reading.

In all careful tests extending over a considerable length of time, made at the University of Illinois, all strain-gage readings are corrected and adjusted for variation of temperature and of personal equation by the use of one or two standard bars. The strain gage is used as a transfer instrument between gage lines on the specimen and the gage lines on the standard bar.

CAUSES OF VARIATION IN READINGS

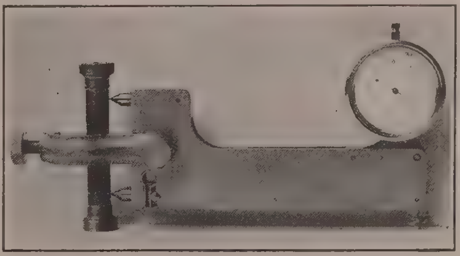
The usual causes of variation in instrument readings with the strain gage are: (1) Variations in length of the strain gage caused by temperature changes arising from handling by the operator, (2) expansion or contraction of the member tested, due to temperature change, and (3) stress set up in a member by structural restriction of expansion or contraction under temperature changes. If a standard bar of invar steel is used, the change of length of the standard bar itself under varying temperature may be neglected; and with an invar steel bar the correction introduced in reducing instrument readings will be for the first only of the three causes mentioned above. If a standard bar of material similar to that in the test piece is used and the temperature of standard bar and member remains the same during a series of readings, the corrections introduced will be for the combined effects of the first two causes mentioned above. No simple method of correcting for the third-named cause has been devised.

As to the accuracy of the strain gage, it was concluded after special studies on a reinforced-concrete building that "under difficult conditions stresses in steel can be determined within ± 500 lb. per square inch, and that under favorable conditions with careful work it may be determined within ± 200 or ± 100 lb. per square inch."

The speed which may be developed in the operation of the strain gage will vary greatly with conditions of the test. In laboratory practice where gage lines are accessible and holes are well prepared it is not uncommon for an experienced observer to take observations on 50 gage lines per hour.

Trussed Concrete Bridge

Reinforced-concrete trusses are used instead of plain beams to support the longitudinal girders of a concrete bridge of the girder type across the river Lagan at Belfast, Ireland. The design is described in a recent number of the "Engineer." The bridge is 30 ft. wide and 195 ft. between abutments and is divided by three pairs of piers into two end spans of about 40 ft. each and two center spans of 50 ft. horizontal clearance. There is a clear transverse span of about 16 ft. between each pair of piers. The piers are of 1:6 concrete, without reinforcement, each pair being connected above high-water level by a reinforced concrete truss, the horizontal members of which are 12 x 12 in. in cross-section. The center vertical and two web members are of the same widths, while two other diagonal members are 12 x 16 in. in area. In the lower chord six ⅞-in. rib bars were used; in the compression members four ½-in. bars were sufficient. The bridge is said to have cost about $12,000.



Berry Type with 2-In. Gage Length

Preventing Erosion of Highway Embankments by a Grass Thatch

On the Sual Hill diversion of the Lingayen-Alaminos Road, in the Province of Pangasinan, Philippine Islands, there is an earth embankment 42 ft. high and 187 ft. long on top. The width of the fill on top is about 13.12 ft. and the greatest width at the bottom is 121.4 ft., with average side slopes of 1.2 to 1. The fill was made and the road surfacing completed just previous to the 1912 rainy season. Notwithstanding the protection offered by the surfacing materials, the erosion of the side slopes was so great that channels were cut to a depth of 6 feet. The earth used in the fill is a rather finely divided white clay which does not pack readily. Sod protection for the immediate future was out of the question, as succeeding deluges might be expected before the grass could become rooted. It was thus necessary to resort to a cogon thatch as an immediate means of preservation after the damage to the fill had been repaired.

The "Quarterly Bulletin" of the Bureau of Public Works, in telling of its use on this work, describes the cogon plant as a perennial grass growing from 3 to 5 ft. high, common in the Philippine Islands and southeastern Asia. It has long, narrow, silvery silky panicles, and is so hardy in its growth and habitat that it smothers out all other plants in its territory. This grass was cut as long as possible, the average being from 4 to 5 ft., and brought to the works in bundles or "manojitos" about 2 in. in diameter. The bundles were laid close together against the side of the fill and the butts brought to a line on the uphill end. Each bundle was fastened down separately by driving a short bamboo hook over each one about 4 in. from its butt into the fill. The courses were begun at the bottom of the fill and carried up the slope, spacing them about 20 in. apart. No wire, poles, stones, or other fastening device except the pegs was used. However, the stability of the thatch was perfectly satisfactory and no wash was detected on any part protected by the thatch.

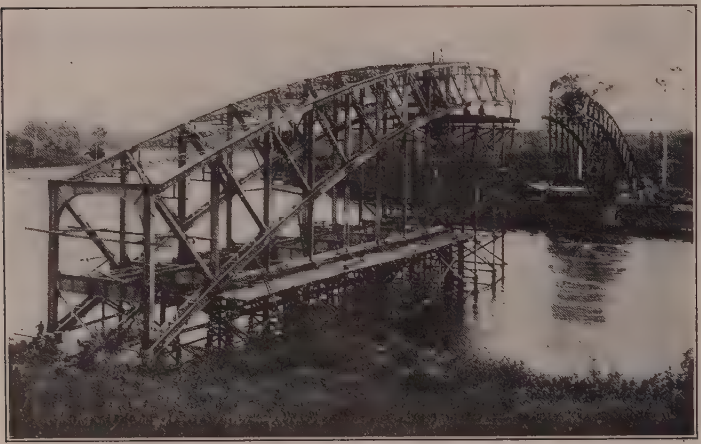
Before the thatch had time to rot away young cogon plants were set out at intervals of from 1 to 2 ft. all over the sides of the Sual Hill embankment. This was accomplished by merely inserting a sharpened stick through the thatch and earth, and planting the cogon.

The area of thatch used on Sual Hill fill was about 1435 sq. yd. Its total cost at Philippine prices for labor was about $27. It is thought that the saving in volume of fill secured by increasing the steepness of the side slopes was more than sufficient to cover the cost of the cogon thatch.

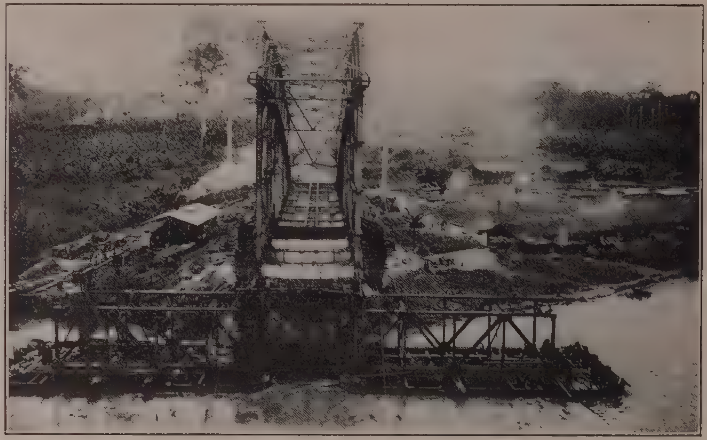
Iron and Steel Railway Bridge in German Cameroon

By Dr. Alfred Gradenwitz, Berlin, Germany

The Cameroon Midland Railway is one of the main lines of that promising German colony and is destined to open up the larger part of South Cameroon for the ocean harbor of Duala. At Edea, about 2000 ft. below the famous waterfalls, this railway crosses the Sanage River, which at that point is divided by an island into a northern and a southern branch. While the former could be crossed by a bridge comprising four or five small spans on river and land piers, conditions are much less favorable in connection with the southern or main branch. The southern river bed, in fact, had already at a normal water level (which generally obtains in the spring and autumn) a water depth of about 60 ft., and during the high-water period in September and October sudden rises are frequently observed. While these circumstances greatly complicated the contemplated construction of a railway bridge, the condition of the subsoil proved to be an even more difficult factor. Special attention had also to be paid to the condition of flow. While being very considerable even with a normal water level, the rate of flow with high water rises to about 40 ft. per second; moreover, there are violent eddies, mainly due, it seems, to the curvature of the river.



Bridge in Course of Erection, Nearing Closure of Arch



Progress View of Bridge, Showing Supporting Pontoons

The Gutehoffnungshütte, of Oberhausen, Rhenania, was intrusted with the supply of the iron superstructure of the bridge; the substructure being designed by the railway company itself, partly on plans by the firm above mentioned. By far the larger number of workmen had to be recruited from the natives and they comprised only a few skilled men. The time available for the construction of the bridge was extremely short. The various parts of the structure, with a view to their transport and the available loading devices, were not to exceed a maximum weight of 2 tons.

The following guiding principles were therefore attended to in designing the bridge: The installation of river piers being undesirable, the bridge was to consist of a single span. On account of the excellent condition of the river banks (gneiss rock) an arched bridge with horizontal thrust was found to be the most suitable solution, the abutments being advanced sufficiently landward to permit of their being constructed in a dry building pit independent of any fluctuations in the water level. The installation of scaffolds in the river was possible only on the left bank, insofar as the sand layers there could be considered as being sufficiently deep.

Basic Thomas ingot iron of normal strength was used as building material for the superstructure, the abutments being made of cast-steel. On account of the increased risk of rusting entailed by the tropical atmosphere, no constructive parts subject to any considerable stress were made less than ⅜ in. in thickness.

The abutments, after leveling the rocky ground, were designed as simple dry concrete blocks. The left half of the bridge was mounted on a scaffold driven into the sand layers. The right half was fitted up on the railway level and pushed on by means of pontoons. The outfit used in this connection consisted mainly of eight iron pontoons, each about 10 ft. wide and 6½ ft. long, combined in two groups of four units each transversely to the bridge axis by means of an iron scaffold bridge 20 ft. wide and 140 ft. long, which was so designed as to be used eventually as a single-track bridge of 140-ft. span. Wire ropes, the winches of which were installed on the opposite bank, were used in hauling the right half of the railway bridge into position.

After this floating process, which lasted about two hours, the two halves of the bridge were tightly drawn together by means of two tackles, thus avoiding any longitudinal fluctuations of the floating parts. The right-hand turning joint was then brought into gear by lowering the bridge about 3 ft., and inserting a wedge. In the meantime, however, the water level had increased sufficiently for the proper level of the vertex to be reached after loading the pontoons slightly with water ballast. The winches on the scaffold having then been released, the pontoons were even further lowered so that the bridge now rested freely in its turning joints. This operation took about three days, after which the missing runway parts and the upper chord having been fitted, the joint was finished.

Hauling Nitroglycerine with a Motor Truck is being done regularly by the Eastern Torpedo Company, of Bartlesville, Okla. A White truck with a special body has been built for the purpose, the nitroglycerine being used for "shooting" oil wells. To guard against explosions several pine cross-sills have been placed across the truck frame; these sills are spanned with a layer of asbestos to keep the engine heat from the load. Above the asbestos is a solid pine floor covered with a thick rubber mat to reduce jolts. On the mat is a copper pan covering the entire loading space and provided with high sides to guard against leakage from any car reaching the chassis.

White River Hydroelectric Plant

First of a Series Designed for the Full Development of the Potential Power of the Stream

The Ozark Power & Water Company is completing its 18,000-kw hydroelectric development on the White River near Forsyth, Taney County, Missouri. The power will be used for lighting and power purposes in Springfield and Joplin and is already sold to the Springfield Gas & Electric Company and the Empire District Electric Company, the power to be furnished at their switchboards. The present capacity of the plant will be five units of 2250 kw each, and there is room for three more of the same capacity. These latter units are expected to be installed shortly after the plant has been put in operation, the market for power being almost unlimited in that part of the state.

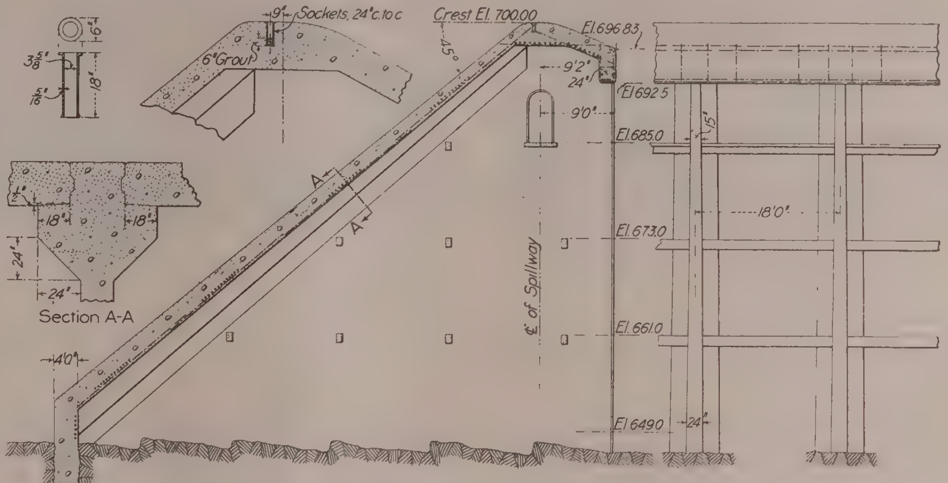
General Conditions

Above the dam the drainage basin of the White River is approximately 4350 sq. miles in area, with a mean precipitation of 40 to 45 in. per year, giving a discharge varying from a minimum of 750 to a maximum of 126,000 sec.-ft. The discharge in the river is not always in proportion to the precipitation, due to the infiltration of water from adjacent drainage basins. That this infiltration can be depended upon for a steady run-off, augmenting the normal minimum discharge if the one watershed alone were tributary, is shown by the fact that in several places the springs are so large that even the warmest summer does not dry them up.

The White River and its tributaries have been surveyed and studied from their sources to the lowest possible development, and plans have been made whereby every foot of fall and every cubic foot of water will be utilized. Ultimately there may be six or eight complete installations, each of which will have a considerable pondage and at the same time act as a storage reservoir for the plants further downstream. One of these developments, on the James River near Virgin Bluff, is now in course of construction and another will shortly be undertaken.

The principal reservoir will be at the upper end of the system and will have an area of about 10,000 acres.

The dam is of the Ambursen type, with a maximum height of 56 ft. The reinforced-concrete deck is supported at an angle of about 45 deg. by vertical tapering concrete buttresses spaced 18 ft. on centers. Solid close-grained limestone, with heavy horizontal strata, forms the foundation. Test pits were sunk to a depth of 10 or 12 ft. and the required depth of excavation was determined. This averages about 5 to 6 ft. A cut-off wall about 8 ft. deep and 4 ft. wide runs the entire length of the dam at the heel of the deck. The downstream side of the dam is open and the water is allowed a free fall after passing the crest. The dam is 594 ft. long. Six openings under the dam and three through the deck permit the river to pass the site during construction. All but two of these openings will eventually be closed with concrete. Two openings under the dam, 5 ft. wide by 5 ft. high, will be fitted with sliding cast-iron gates and serve as permanent sluiceways. These gates will be operated from a gallery inside the dam. Beyond the west abutment, which is 25 ft. higher than the spillway crest, is an earth dike about 600 ft. long, with a cellular core wall of reinforced concrete.



Cross-Section and Details of Dam with Flashboard Sockets

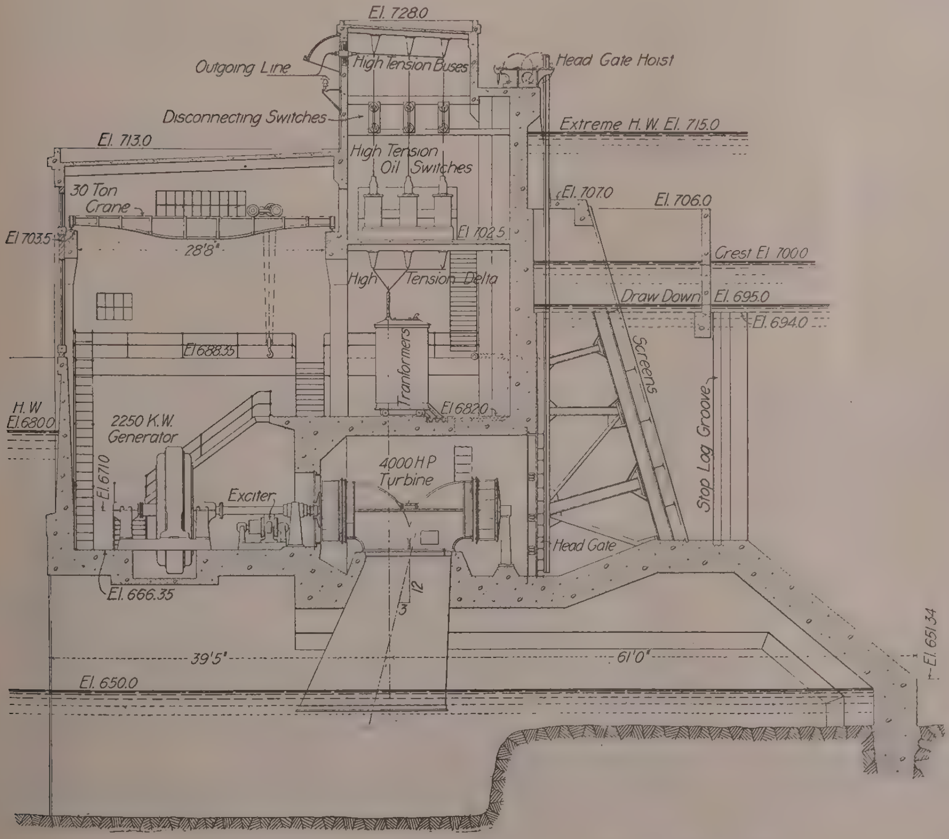
The dam was completed ready for closing just inside of the time limit. The last concrete was placed in the deck of the dam on March 23. It was no sooner placed than the heavy rains which swept over the whole country at that time commenced, and while this concrete was still green a telephone message was received from the headwaters that the river was rapidly rising. It became evident at once that the dam must be closed at all hazards, otherwise the accumulation of heavy logs, trees, etc., that would be swept down by the flood would jam in the closure openings and prevent the closing of the dam for several months, involving also expense of clearing out the debris. The alternative was accepted of risking the comparatively green concrete, then only two days old. The gates were accordingly dropped, and twenty-four hours later there was a depth of water of 6½ ft. on the crest of the dam.

The dam was found to be free of leakage and showed no defects whatever even under the severe conditions of closure. The extreme high water went through the power house, which was not wholly completed. It did no damage, however, and work was resumed as soon as the crest of the flood had passed.

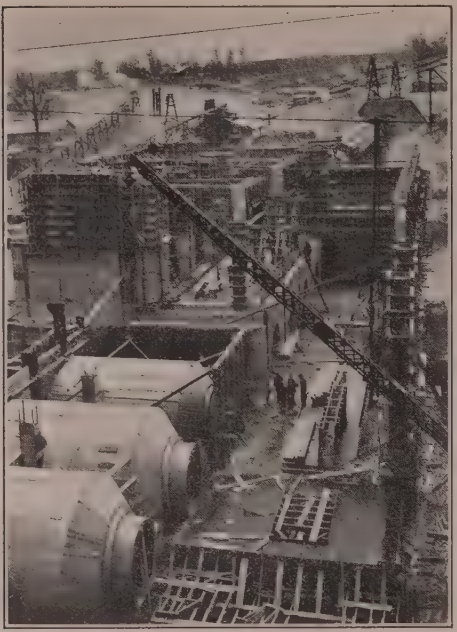
Although the dam was closed in less than the specified time it would have been closed at least one month earlier except for delays in the shipment of the hoisting gear for the gates.

Power House and Intake

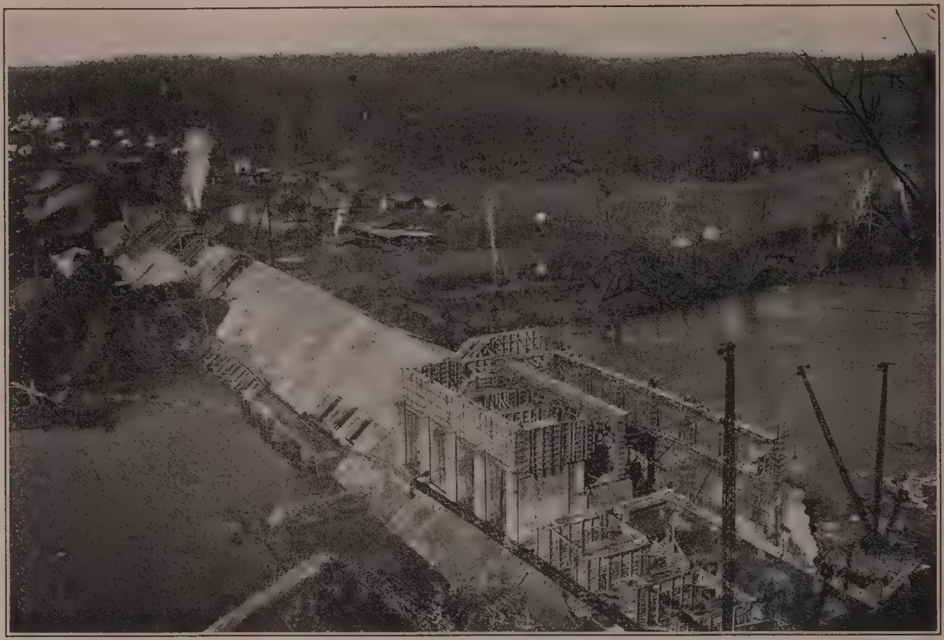
The power house is located at the east end of the dam and built integral with it. It is located in the main channel of the river, which has been diverted during the construction. A stone-filled crib cofferdam served to keep the water out while the work on the foundations was going on. The cribs were



Cross-Section of Generating Station on Axis of Main Unit



Form Work for Turbine Chambers



View of Spillway and Power House on Jan. 16, 1913

made of rough timbers, drift bolted together and sunk in the river with free spaces between them. These spaces were later filled by smaller cribs and the outside face of the cofferdam was sheeted with double courses of 1x12-in. rough boards, breaking joints. Outside of this sheeting was an earth dump. This cofferdam proved to be remarkably tight and only a small amount of leakage was observed. No water-bearing seams were found in the rock. Material in the power house is reinforced concrete throughout.

Approaching the wheel chambers the water flows first through one opening 23 ft. wide by 18 ft. high for each wheel at a velocity of 1.6 ft. per second. The upper edge of these openings is submerged to a depth of 4 ft. at low water, thus skimming the water of floating debris and ice. Stop-log slots 12 in. wide and 6 in. deep, with wooden tightening beams at the bottom, are provided so that these openings may be conveniently closed.

The partition walls between these intakes are, therefore, extended 4 ft. outside of the power house and tapered in plan.

The screens placed in back of these openings are inclined on a slope of 3½ horizontal to 12 vertical. The width of screen is 18 ft. and length of bars 40 ft. The bars are of 2¾x¼ in. cross-section, spaced 1½ in. on centers. Beams, five in number, support these bars at intervals varying from 4 to 7 ft. The loads from the beams are carried to the power-house structure by means of two heavy trusses with channel-bar members. The velocity of the water through these screens is normally 12 ft. per second and the decrease in intake area due to the screen is 16 2-3 per cent.

The head gates are 17 ft. high by 19 ft. wide, made of steel consisting of ⅜-in. skin plate supported on 8-in. beams. At the bottom of the gate is a 6x8-in. timber, which tightens against the back of a 10-in. channel placed in the concrete across the opening. The gate frames and guides are made of rolled channel and beam sections.

A 60-ft. stem is necessary to operate the gate. It is made up of a 10-in. channel reinforced by a 6-in. beam, and at the top is a rack 20 ft. long engaging a pinion on a hand and motor driven hoist. This hoist is so designed that the gate can be pushed down if necessary.

The operating platform for the head gates is 7 ft. wide and runs the entire length of the power house at an elevation 5 ft. above extreme high water.

TURBINES

Eight horizontal, twin, Francis turbines and two single-runner exciter turbines made by the S. Morgan Smith Company will complete the ultimate installation. Of these, five main units and the two exciters are now being placed. The wheels are designed for a normal head of 50 ft., have two 51-in. runners and will operate under a normal speed of 214 r. p. m. These wheels are designed for a capacity of 4000 hp, but will normally be operated at 3000 hp. They are direct-connected to 2250-kw, two-phase, twenty-five-cycle, 2300-volt General Electric generators, designed for 30 per cent overload, thus giving a capacity approximating that of the wheels. The discharge through the turbines will normally be 660 sec.-ft. and is taken care of, after having left the runners, in a single conical steel draft tube, flaring from a diameter of 9 ft. 6 in. to 17 ft. This draft tube has its axis inclined downstream on a slope of 1 horizontal to 4 vertical, thus giving the water a horizontal velocity component, facilitating its discharge from the tailrace chamber and avoiding obstruction of the tailrace, and the raising of the level several inches. The velocity at the mouth of the draft tube is normally less than 3 ft. per second.

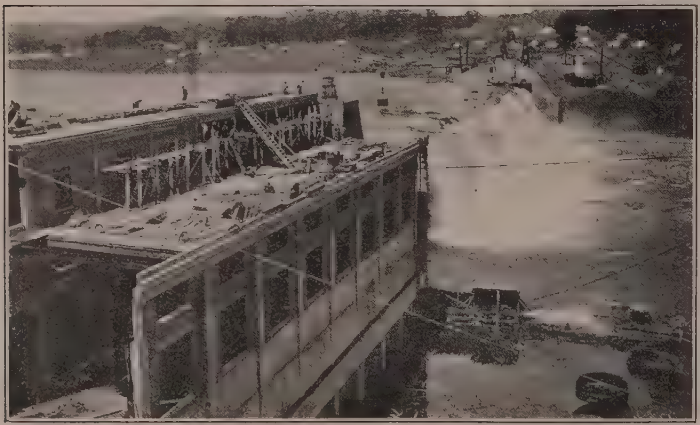
The exciters are driven by single-runner S. Morgan Smith turbines in cylindrical steel casings, with quarter turn and steel draft tubes.

ELECTRICAL EQUIPMENT

The main units are provided with Lombard type N. S. governors, the governor pump being belt-driven from the shaft of the unit. This governor is of the closed type and operates under 200-lb. pressure and has a cylinder diameter of 10 in. and a stroke of 24 in.

It is capable of developing 30,000 ft.-lb. per second.

From the generators the current is led

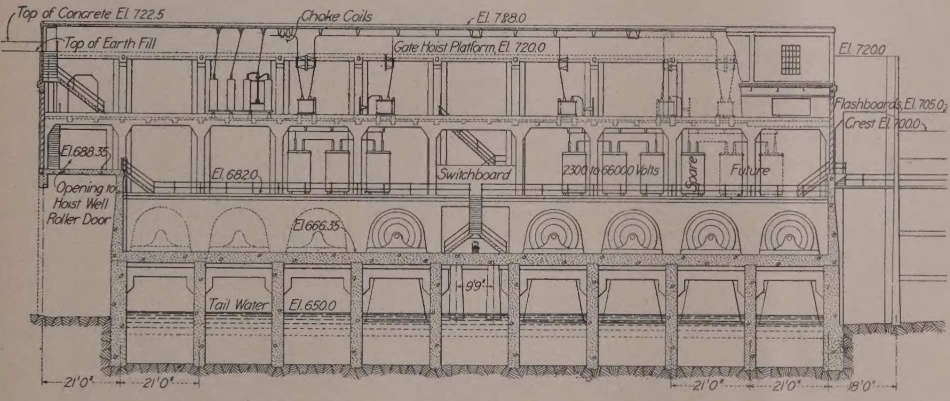


View of Power House and Completed Spillway

Downstream View of the Buttressed Spillway through low-tension switches to transformers and stepped up from 2300 to 66,000 volts. Low-tension switches, switchboard and the step-up transformers are all located on a gallery 15 ft. 8 in. above the floor, so that the switchboard operator has a clear view of the generator room. Ease of operation with few attendants is secured by placing all of the machinery on one floor, thus doing away with single and double deck tunnels below the generator floor. The high-tension switches and buses are isolated on a floor 20 ft. 6 in. above the switchboard gallery.

There is at present but one circuit, leading from the power house to Springfield and Joplin. The first 26 miles are carried on steel towers of the A-frame type, with four-legged anchor and corner towers. This part of the line goes through broken country, and the span is, therefore, only 500 ft. The balance of the line is provided with 40-ft. Idaho cedar poles having 8-in. tops. This part is about 114 miles long and has steel strain towers.

At Joplin the line connects with the 33,000 volt transmission system of the Empire District Electric Company. The present steam stations in Springfield and Joplin will be maintained and act as reserve plants with a combined capacity of about 20,000 kw. Delivery of power is based on 60 per cent load factor, being the same as for the present steam stations.



Sectional Elevation of Generating Station; Dam at the Right

Most of the materials have been hauled from Branson, Mo., to the power site over an 8-mile road through broken country. Wagons were, therefore, driven in pairs so that the teams could help each other up the steeper grades. Branson is located on the White River above the dam site, and when the river permitted navigation—that is in flood seasons—the materials were transported in barges. As soon as the dam is completed and the reservoir filled, the machinery, now being stored at Branson, will be transported in barges to the power house. All of the required materials and machinery have been taken to Branson over the Missouri Pacific Railroad.

The material in the dam has mostly been placed by two cableways, and for the powerhouse work the cableways have been supplemented by two structural-steel, stiff-leg derricks. The mixer plant is located on the west shore and has inclined trestles leading up to its bins. All sand has been dredged from a bar in the river below the dam and all gravel from a bar above. The sand is of unusually good quality, being fine and sharp.

The work has been carried on during 1912 and was completed about April 1, so that the costs are now known. The estimated cost for the Ambursen part of the job—for the dam and power house with appurtenances, except machinery—was $450,000 and was guaranteed under all circumstances not to exceed $500,000. The actual cost is $440,000, or about 2 per cent less than the original estimate. The appropriation for the whole plant is $1,500,000, but the cost will be a little less.

The Ambursen Hydraulic Construction Company has designed and built the concrete work in the plant, which is financed by the Henry L. Doherty Company, of New York. Mr. Henry L. Doherty is president of the Ozark Power & Water Company and Mr. T. O. Kennedy is engineer in charge of the construction. Mr. C. E. Carter is in charge of the transmission lines and Mr. George Saattoff is in charge of the electrical equipment. Mr. George Hayler will be in charge of the operating company after the plant is completed.

Hammer Drills in Highway Construction

The construction of road 5266 of the New York State highway system in Jefferson County requires the excavation of about 5000 cu. yd. of rock in what is known locally as the Depauville cut. At this point the road, which is of water-bound macadam, passes over Graber Hill, the grade formerly being 10 to 12 per cent, but by making a cut of more than 25 ft. at its maximum depth the grade was reduced to approximately 4 per cent. The soil of Graber Hill is underlaid at a depth of about 2 ft. with a 7-ft. stratum of gray limestone. The limestone is unusually hard and tough and is interlaid at intervals of approximately 1 ft. with seams of hard clay from ½ to 3 in. in thickness. The removal of the limestone by ordinary methods was found to be exceedingly difficult.

A tripod drill operated in connection with a stationary steam plant was first tried by the contractor, but with poor results. The seaminess of the rock interfered with drilling by causing the drills to bind, necessitating more frequent blasting. This necessarily slowed up the work and made it more costly, as more powder had to be used and the drilling equipment had to be removed and reset more often. Looking for a solution of the problem the contractor experimented with hammer drills operated by compressed air. The result was considered satisfactory from both operating and cost points of view.

While hammer drills are in general use in mining operations, their adaptability to road work is not widely known among highway contractors and engineers. The drill employed on this work was a large type of the common hand drill and used a hollow steel of hexagonal shape through which air was discharged for the purpose of removing the dust and stone cuttings under the bit of the steel. The contractor reports that he has since used an automatically rotating drill in similar rock work with even better results.

Although the average depth of the holes was about 8 ft., holes were occasionally drilled to as great a depth as 12 ft. in sound stone without the binding action being noticeable. A short steel or starter, from 1 to 1½ ft. long, was first used with a bit considerably larger in diameter than the steel itself. Successive steels, each about 1 ft. longer than the preceding, and each with a bit slightly smaller in diameter, were used until the desired depth was reached. In the early operations with the tripod drill the bit had frequently been forced through thin layers of rock under the action of the heavy blows to which the drill was subjected in much the same manner as an auger penetrates a thin piece of wood. This tended to reduce the diametral dimensions of the hole, make it irregular in shape and bind the drill. The lighter and more frequent blows of the hammer drill, which is dependent on its own weight and not on forced cranking for its feed, gave it a tendency to rebound and eliminated the binding action when penetrating alternating hard and soft strata.

Another advantage observed was the ease in setting up. Very little time was required to move from one point to another and as much as 240 ft. were drilled by two hammer drills, each operated by one man, in a day. Where a hole close to the embankment was necessary to distribute the shot properly, the hand drill could be started with no more preliminary trouble than if the shot was to be placed in the center of the cut. There were no limitations in its use as to inclination of hole, but a maximum depth attainable with only one drill operator was 12 ft. A greater footage of holes was drilled with the hammer drill than was economically possible by the other methods, and with a greater number of holes the rock was broken into smaller pieces and of better size for removal in dump carts.

The hammer drill was connected to an Abenaque portable air compressor outfit which comprised a 20-hp. horizontal, four-cycle, gasoline engine mounted on a steel frame and direct-connected to an Ingersoll-Rand air compressor. The whole was mounted on wheels and followed closely the work of the drillers as they progressed through the cut. When not in use on the drilling work the outfit was moved to a nearby rock crusher, the compressor disconnected, and the engine belted over a friction clutch pulley to the crusher and employed in breaking the rock used in the foundations for the macadam construction.

Water for the work had to be drawn in tanks from Depauville, 2 miles distant. A saving in cost of this account was noticeable as compared with steam-operated drills, as the outfit required only 2 bbl. of water a week for use in the cooling system of the engine. Approximately 18 gal. of gasoline were required for one ten-hour day run.

The work was done by Coleman Brothers, general contractors, of Boston, Mass.

A 192-FT. SPAN, single-track railway lift bridge, with a maximum clearance of 55 ft., has recently been built over the Mississippi River for the Chicago Great Western Railroad. It replaces a 412-ft. drawspan built in 1885 and reinforced in 1910 and is of the Waddell-Harrington type, with 225-ton concrete counterweights. The Widell Company and the American Bridge Company were the contractors for the substructure and the superstructure respectively, which were built under the direction of Mr. L. C. Fritch, chief engineer, and Mr. C. Chandler, engineer of bridges.

Pivot Pier of Chelsea Bridge North

Hollow Concrete Shaft and Foundations Built in Submerged Basket Crib

Chelsea Bridge North carries highway and street-car traffic over the north or main channel of the Mystic River, between the Charlestown District (Boston) and Chelsea, and forms a part of one of the most important highways out of Boston. It has pile approaches, built in 1880, and had a retractile drawspan built in 1895 and lengthened in 1900 to span a waterway 60 ft. in width.

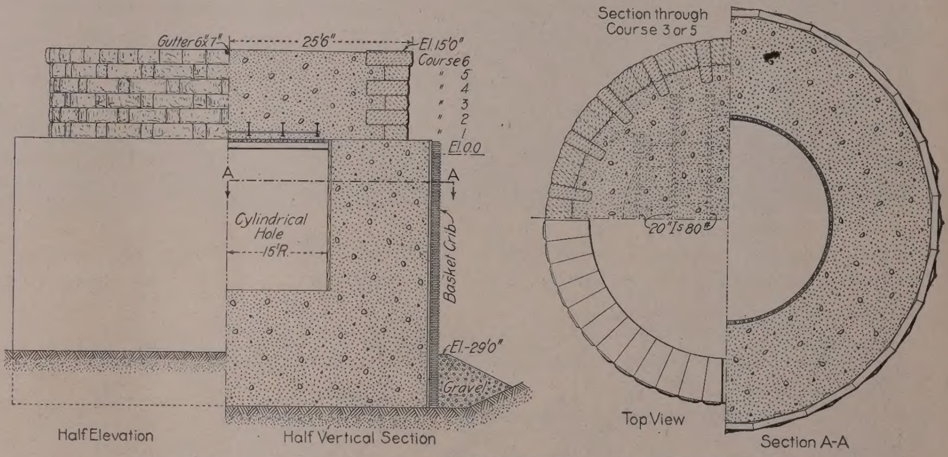
On account of the rapidly increasing water traffic of the Mystic River, and because the waterway was in an unsatisfactory location, the Secretary of War, on January 3, 1910, ruled that the bridge was an unreasonable obstruction to the free navigation of the river and ordered that the clear width of the draw opening be increased to 100 ft. or more.

It was at first proposed to provide for traffic for a period of fifteen or twenty years by the construction of a temporary bridge at one side of the existing structure, with a single-lift drawspan over a channel 125 ft. in width. At that time practically the whole cost of reconstruction and maintenance of the bridge fell upon the city of Boston. In 1911 the legislature provided for the apportionment of the cost of the work among the city of Boston, a street railway company, the city of Chelsea, and other cities and towns benefited. With the passage of this legislation and a request from shipping interests for two waterways instead of one the plan for a temporary bridge with a bascule drawspan was abandoned, and instead it was decided to build a new swing drawspan on a masonry pier, the whole to form a part of the complete permanent steel and masonry bridge to be constructed later.

The new drawspan is to afford two waterways, each 125 ft. wide in the clear and 30 ft. deep below low water. It will be 363 ft. long over all, 60 ft. wide, and will weigh 1400 tons, affording a clearance above mean low water of 25 ft. The contract for building the permanent pivot pier and the wooden fender piers, for the rebuilding of the existing pile approaches, and for building and removing the temporary by-pass bridge to provide for travel during the reconstruction was awarded in February, 1912, to Mr. George T. Rendle, of Boston, and will cost approximately $200,000. The estimated total cost of the whole work, including the draw superstructure, is $425,000.

Pivot Pier

The pivot pier has a concrete base 60 ft. in diameter and 39 ft. high, with its foundations on solid rock and the upper part about 1 ft. above mean low water. The weight of the base and the volume of concrete required for it are reduced by the construction of a concentric cylindrical chamber 30 ft. in diameter and 22 ft. high in the upper part. The top of this chamber is spanned by steel I-beams supporting part of the load from the pier shaft above, which is 51 ft. in diameter and about 13 ft. high from the top of the base to the top of the coping. It is faced with five courses and a coping of quarry faced granite, backed by a solid mass of concrete, which, like that of the base portion, is made of 1:2:4 Edison Portland cement, sand, and stone up to 2 in. in diameter.



Pivot Pier, with Concrete Deposited under Water in Permanent Crib

At the pier site an excavation 65 ft. in diameter was dredged to bedrock, which was found at depths varying from 30 to 38 ft. below low water. The material was removed chiefly by a dipper dredge and consisted of about 8 ft. of soft silt mixed with sand, from about 17 ft. to 25 ft. below low water level, beneath which there were about 8 ft. of blue clay, and then a stratum of very hard sand, gravel and clay, with a thin stratum of shale covering the bedrock. The slopes of the excavated area were maintained at 2½ horizontal to 1 vertical.

Basket Crib

The basket crib, or form for the pier foundation, was built of about one hundred and forty-five horizontal courses of 3 x 12-in. yellow-pine planks, 8 ft. long, laid flat and breaking joints. The ends were beveled to make radial joints, and each plank was secured to those below it by 1-in. oak treenails 9 in. long, two at each end of each plank, about seven thousand eight hundred treenails being required for the entire crib. In addition the planks were well spiked to the lower courses throughout their entire length with 6-in. spikes. The courses were also secured together by 4 x 12-in. vertical planks opposite alternate joints, which were fastened to the inner circles of the crib by lag screws. The crib, which contained about 83,000 ft. b. m. of yellow-pine plank, was built up to a height of 3 or 4 ft. on shore between high and low water marks, and was then floated to deeper water and completed while still floating. After completion it was towed into position and held by guide piles, spaced about 12 ft. apart around its circumference.

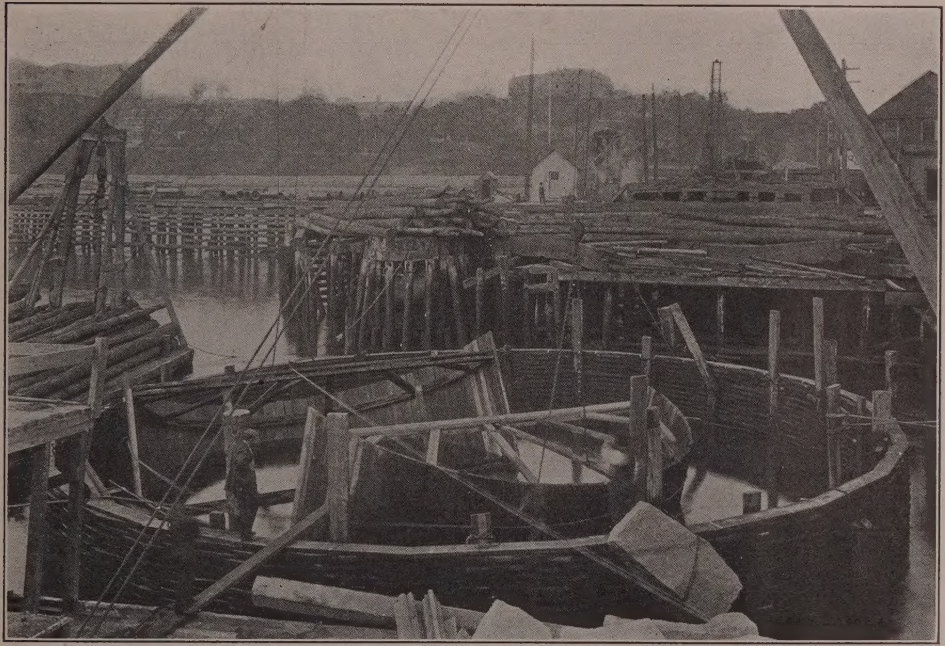
To sink the crib it was first planned to build exterior pockets which would be filled with gravel, but they were eventually dispensed with, and the crib was sunk by loading it with old iron, with stone intended for use in the pier masonry, and with heavy chains hung over the walls of the crib.

There was no attempt to construct the crib so that on the bottom it should conform to the variations of the rock surface. Instead, the bottom of the crib was made level and it was sunk until it took bearing on only a portion of the lower edge at the highest rock level. Then, to provide continuous bearing at all parts of the circumference, and especially to complete the inclosure of the crib and confine the concrete that was afterward deposited within it, wooden boxes of varying size, but averaging about 4 ft. square, and 4 ft. deep, were filled with lean concrete, lowered to the bottom and placed by divers under the edge of the crib to form a continuous wall. After the concrete boxes were placed the excavation outside of the crib was backfilled with gravel and dredged material until the whole crib was surrounded by filling to about 29 ft.

Girders Set over Chamber in Pier; Concreting above Water Level below low water, or some 2 ft. above the bottom course of plank of the crib. This backfilling formed an effectual seal to retain the concrete which was deposited in water inside the crib without unwatering the latter.

CONCRETING

Broken stone and sand were delivered both by lighter and by team. The cement was conveyed by team. All the materials were stored on the old bridge close to the draw opening, where the stone and cement were measured by shoveling into scale boxes. Between the old bridge and the pivot pier was moored a concrete mixing scow, equipped with a Milwaukee mixing machine and a boom derrick. The derrick delivered the scale boxes of sand and stone from the old bridge to the charging hopper of the mixing machine. The concrete was mixed in 1-yd. batches and delivered to a submarine bucket with bottom flap doors. The bucket was handled by the derrick on the mixing scow and was so constructed that the doors were not opened until the bucket was seated on the bottom of the pier foundation, after which the concrete was automatically deposited as the bucket was lifted by the derrick.



Placing Forms for Pier Chamber on Submerged Concrete

After the concrete foundation had been built up to within 20 ft. of low-water level a permanent form was set in the center of the pier for the cylindrical chamber above referred to, and the remainder of the concrete was deposited in a concentric ring between this form and the basket crib. The concrete was deposited in water up to about low-water level, and as the basket crib extended about 5 ft. above low water it was possible to unwater the crib at half tide or less, and the remainder of the work on the pier was carried on in the dry. This consisted of placing 20-in. steel I-beams over the hollow center, the building of forms upon these I-beams, and the covering of the whole pier foundation with a layer of concrete about 2 ft. in depth, which was leveled off in readiness for the laying of the stone masonry of the upper portion of the pier.

The pier contains about 3673 cu. yd. of concrete and 323 yd. of granite masonry. The construction of the basket crib was commenced Aug. 8, 1912. It was sunk in position Sept. 14, 1912. Concreting was commenced Sept. 28, 1912, and the pier was completed ready to receive the superstructure Dec. 7, 1912.

The work was designed and executed under the direction of the Public Works Department of the City of Boston, of which Mr. L. K. Rourke is commissioner; Mr. Frederic H. Fay, engineer, Bridge and Ferry Division, and Mr. S. E. Tinkham, engineer of construction.

Pure and Wholesome Water

In a paper on the subject "Pure and Wholesome Water," by Mr. George A. Johnson, read before the Minneapolis convention of the American Waterworks Association, the weakness of sanitary science in its efforts to interpret the significance of the data obtained in the examination of water was laid bare. Mr. J. W. Ellms, in discussing the paper, said that he sympathized to some extent with Mr. Johnson's pessimism as regards the unsatisfactory state of the knowledge on this subject and the inability of sanitarians to prove mathematically whether a water is or is not pure and wholesome. Nevertheless, it appears to Mr. Ellms that a more optimistic view can be taken of the matter. It can hardly be denied that sanitarians are constantly distinguishing between pure and impure waters and that they do so successfully despite the varied and complex character of the data set before them. Mr. Ellms states that no satisfactory discussion of this question is possible unless there is an agreement upon the definition of a "pure and wholesome water." Mr. Johnson, he believes, has correctly defined what is a common understanding of sanitary purity for a drinking water, viz., "water which is free from noticeable turbidity and color, offensive tastes and odors, and forms of bacteria known to produce disease." So far as the tests which are used to detect objectionable physical properties are concerned, he believes them to be fairly adequate for the purpose. Natural waters may possess undesirable properties in varying proportions, and toward them different persons will have varying degrees of aversion. Herein lies the difficulty of strictly defining the limits for those qualities which offend the esthetic sensibilities of the individual.

The value of many of the chemical tests taken separately is slight, but considered collectively and in combination with the information supplied by the bacteriological and physical tests, they throw much light on the past history and present condition of the water. Certain of these chemical tests, Mr. Johnson stated, are invaluable, especially where the operation of purification plants is dependent upon a knowledge of the amounts of certain constituents, as, for example, the degree of alkalinity of a water in plants employing chemical coagulants. Even the nitrogen content of the water, knowledge of which Mr. Johnson seems to believe is particularly useless, Mr. Ellms believes is not without value in classifying a water and determining its general character.

From the hygienic standpoint the determination of the purity and safety of a water for drinking purposes is most difficult. The inadequacy of bacteriological methods must be admitted. However, there is no reason for discarding the only available methods for obtaining information, merely because these methods do not furnish all the knowledge that is desirable. Most of the evidence obtained is purely circumstantial. Bacteriological technique does not permit of easy and rapid methods for the isolation of disease-producing organisms. Nor is it by any means certain that all of the water-borne diseases are known. Pathological conditions in the human body are probably produced by organisms, of which we at present have no knowledge. These organisms may have water as their natural habitat, or they may become adjusted to such an environment. Hence the difficulty of establishing the purity of a water unless our technique provides means for the identification of all known pathogenic organisms. In lieu of direct proof our ignorance forces us to resort to circumstantial evidence, which often is of a most convincing character.

Mr. Ellms states that he has a great deal of sympathy for Mr. Johnson's attitude toward the sanitary significance, or, rather, lack of it, attached to B. coli. This organism is probably harmless in itself and yet its frequent association with other "vicious and disorderly" bacteria has given it such a bad name that it often suffers on account of the bad company it keeps. Its presence is not prima facie evidence that disease-producing organisms are associated with it. Nevertheless, its presence in a water, when taken in connection with evidence obtained from other sources, can usually be interpreted properly and with a fair degree of correctness.

The chemical, bacteriological, physical, and microscopical data obtained by the usual methods of examination do afford a basis, he thinks, for forming a fairly correct opinion as to the sanitary quality of a water. If the knowledge gained by these tests is supplemented, as it always should be when possible, by information relating to the sanitary conditions surrounding the source of the supply, then one is possessed of all the knowledge about the water that can be obtained. By intelligent interpretation of this information a fairly correct judgment of the character of a water is possible.

A RAILROAD CUT THROUGH COAL on the new construction work of the Louisville & Nashville Railroad in the northern part of Alabama has been converted into a profitable mine by the contractor, who is selling the coal to his neighbor contractors.

Book Reviews

"Manual for Engineers," compiled by Prof. Charles E. Ferris, of the University of Tennessee, appears in its eighteenth edition in slightly revised form. The book is essentially a ready reference compendium of miscellaneous engineering tables and other data, bound in vest-pocket size. The additional material in this latest edition includes instructions for resuscitation in case of electric shock and first-aid methods for injured shop workers. (University of Tennessee Press, Knoxville, Tenn., 50 cents.)

"The Determination of Internal Temperature Range in Concrete Arch Bridges" is the title of a bulletin by Mr. C. S. Nichols, assistant director of the engineering experiment station of Iowa State College, Ames, Iowa, and Mr. C. B. McCullough, assistant engineer of the Iowa State Highway Commission. The bulletin gives the methods used in conducting experiments on the Squaw Creek Bridge at Ames and the Walnut Street Bridge at Des Moines, to determine the yearly range in temperature and the comparative values of the factors affecting this range. Curves are reproduced showing the effect of temperature on the elevation of the crown and the general conclusions are summarized. The Squaw Creek Bridge is a typical reinforced concrete highway bridge with arch spans of about 45 ft. The Walnut Street Bridge was described in the Engineering Record of May 4, 1912, page 492. The field tests made on it were conducted by Mr. W. D. Maxwell, office engineer for the city of Des Moines. (Ames, Iowa, Iowa State College.)

RAINFALL, RESERVOIRS AND WATER SUPPLY. By Sir Alexander R. Binnie, Past President, Institution of Civil Engineers. Cloth, 6 x 9; 148 pages; 56 illustrations. New York, D. Van Nostrand Company, $3 net.

This book, by a British engineer of long and varied experience in Britain and India, contains an expression of the author's views on many important points connected with waterworks engineering. As the author states in the preface, the book is not a complete treatise on this branch of engineering, but is intended to illustrate some important points connected therewith.

The book is based upon a course of lectures given by the author during 1912. As might be expected, some subjects are treated much more completely than others. Probably the experienced engineer will find the chapters on rainfall and on the design and construction of storage reservoirs of most interest and value. The other portions of the book, which deal with the quality of water supply, purification of water, conduits and pipes, distribution systems and various appurtenances connected with waterworks, are less complete, but are interesting as indicating standard British practice.

The author discusses rainfall and runoff in a very interesting manner, giving a review of his earlier papers upon the subject. His long study of this subject makes his views of much value. It is somewhat disappointing to find that the matter of the amount of storage required for different conditions is not more fully discussed and that the tables of data are not brought up to date.

The chapters on storage reservoirs contain an admirable discussion of the construction of earth dams. The failures which have occurred in these structures are reviewed and the reasons of the author's preference for certain types of design and methods of construction are clearly stated. Much information as to selection of reservoir sites is given. The design and construction of waste weirs, reservoir outlets, flood channels and other structures relating to storage reservoirs built with earth dams are well treated.

The book is well written and illustrated, is practical rather than theoretical and should prove of interest and value to all engineers who have to deal with the problem of water supply or with the construction of earth dams for other purposes.

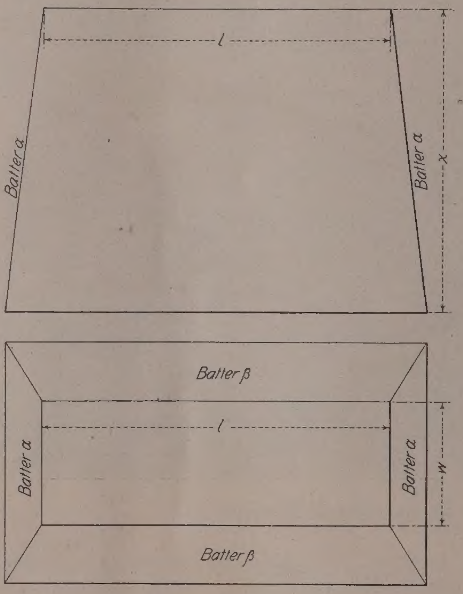
Letters to the Editor

Prismoidal Formulæ to Suit Special Cases

Dear Sir: Prismoidal formulæ by which the contents can be computed with less labor than by using the general formula

$$S = \frac{A + 4M + A'}{6 \times 27} L$$

can often be derived for special cases. In this formula S denotes the contents in cubic yards; A and A', the end areas in square feet; M, mean area in square feet, and L, length in feet between parallel faces. Such a special case is that in which the top area and side batters of a block of masonry are constant and the estimator is required to compute the contents for a number of different heights.



Prism for Deriving New Formulæ

Although the following equation is derived for a special case, it suggests ideas for similar derivations to meet other conditions.

In the accompanying diagram assume that we have given: w and l, width and length in feet of the upper surface or undercoping; α and β, the end and side batters. The contents of the rectangular prism are $w\,l\,x/27$ cu. yd. and of the four wedges, $(\beta\,l + \alpha\,w)\,x^2/27$ cu. yd. The pyramid formed by combining the four corners equals $(4\,\beta\,\alpha\,x^3) \div (3 \times 27)$ cu. yd. Therefore the total contents of the prismoid are

$$\frac{w\,l\,x}{27} + \frac{(\beta\,l + \alpha\,w)\,x^2}{27} + \frac{4\,\beta\,\alpha\,x^3}{3 \times 27}$$

This is of the form

$$y = a\,x + b\,x^2 + c\,x^3 \qquad (1)$$

in which a, b and c are constants. These constants can be computed and a table for any number of different heights readily constructed with the aid of a slide rule and a table of squares and cubes.

If the above prismoid is placed upon a footing course with constant offsets and of constant thickness the equation for the contents of the latter is found in like manner to have the form

$$z = K_1 + K_2x + K_3x^2 \qquad (2)$$

where K_1 K_2 and K_3 are constants.

Adding equations (1) and (2) we obtain,

$$y + z = K_1 + (a + K_2)x + (b + K)x^2 + cx^3 \qquad (3)$$

This has the same form as equation (1), so that if desirable the total contents (body + footing + coping) can be found by solving one equation if we add to K in equation (3) a constant to include the coping.

Toronto. WM. T. CASE.

Making Building Inspectors by Fiat

Dear Sir: Herewith is an interesting clipping from a recent issue of the San Francisco "Examiner," which tends to throw a little light on the subject of poor building inspection.

Alameda is a city of 25,000 population.

H. L. MUCHEMORE.

San Francisco.

The clipping sent by Mr. Muchemore reads as follows:

"After several weeks' deliberation the Alameda City Council agreed yesterday to have the sanitary inspector also be the building inspector, and the Board of Health approved the decision.

"'But I don't know anything about building inspections,' said Eugene Maillot, who holds the office. 'I know all about plumbing and the extermination of flies, and garbage, but nothing about skyscrapers.'

"'We will pay you $10 a month more for the new job—now go and learn all about buildings,' was the answer.

"It is said Maillot has enrolled in a correspondence course in building construction and mathematics to fit himself for the job and rightly earn his extra $10."

A RECLAMATION PROJECT FOR BOMBAY is described in a recent issue of the Engineering Supplement of the London Times. The scheme contemplates reclaiming 1145 acres, stretching some 3½ miles from Colaba Point to north of the Marine Lines Station. A sea wall will be built, running on a continuous easy curve, and the material used for the filling will be mud and clay dredged from the harbor bed contiguous to the proposed reclamation. The steam-power machinery which it is proposed to use is expected to remove the material from a maximum depth of 70 ft. below water at the rate of 2000 cu. yd. per hour and deliver it at any state of the tide to any part of the proposed reclamation. The part of the sea wall exposed to the heaviest attack of the sea will be built upon basalt rock, and will be of concrete faced with basalt masonry; the rest will be of rubble, protected with riprap of 25-ton concrete blocks, and capped with concrete after the rubble has been allowed to settle. The reclaimed area will have an elevation of 2.84 ft. above the highest tide on record, being kept low both to save in filling and to lessen the cost of building foundations. The development plans are provisional, but it is expected to use the most approved principles of modern town planning as adapted to tropical conditions. The trial scheme sets aside 40 acres for a new government house and 150 acres for a public recreation ground, and allowing for wide avenues, streets, open spaces and gardens this leaves some 671 acres for building purposes.